PB-217 949

FLOOD AND POLLUTION CONTROL. A DEEP TUNNEL PLAN FOR THE CHICAGO LAND AREA

Metropolitan Sanitary District of Greater Chicago
Chicago, Illinois

May 1966

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PB 217 949

A PREFEASIBILITY REPORT BY
HARZA ENGINEERING COMPANY
AND
BAUER ENGINEERING, INC.

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PREPARED FOR
THE METROPOLITAN SANITARY DISTRICT OF
GREATER CHICAGO
UNDER
FEDERAL WATER POLLUTION CONTROL ADMINISTRATION
RESEARCH AND DEMONSTRATION GRANT
MAY 1966

HARZA ENGINEERING COMPANY
CONSULTING ENGINEERS
RIVER PROJECTS
400 WEST MADISON STREET
CABLE ADDRESS 'HARZENG CHICAGO' CHICAGO, ILLINOIS 60606 TELEPHONE RANDOLPH 6-3451

May 20, 1966

Mr. Vinton Bacon, General Superintendent
Metropolitan Sanitary District of
Greater Chicago
100 East Erie Street
Chicago, Illinois 60611

Subject: Proposal for Engineering Services
Chicagoland Deep Tunnel Project
Phase B - Feasibility Studies

Gentlemen:

→ The report discusses

We are pleased to present our proposal for engineering services to perform feasibility studies for the initial stage (Calumet Area) of the Deep Tunnel Plan for flood and pollution control in the Chicagoland Area. Feasibility studies should follow as the next step in the planning procedure, as recommended in our prefeasibility report dated May 13, 1966.

Description of the Plan

The initial stage (Calumet Area) of the Deep Tunnel Plan is described in detail in the Prefeasibility Report. Basically, the Plan would utilize existing lateral and main sewers, with additional main sewers to relieve laterals at their high ends. Both the existing and new main sewers would be intercepted at appropriate locations by vertical shafts connecting to a tunnel system 600 feet or more below the ground surface. These tunnels, or galleries, would both temporarily store storm water overflows and drain these waters to a reversible pump-turbine plant location near the Calumet Sewage Treatment Works. Here a reversible pump-turbine plant would pump the water to an upper reservoir. The process of sedimentation, screening and oxygenation that would be inherent in the pump-turbine operation probably would afford adequate treatment to the polluted water, but, if needed, the water could be passed from the upper reservoir through an existing treatment plant before discharge at controlled rates into the Calumet River.

The details of the Deep Tunnel Plan will be developed in the feasibility report but, as presently visualized, the Plan would provide storage space for flood waters in the underground galleries and in the upper reservoir. Storage would be provided to impound runoff from the most severe storm of record, which has an indicated recurrence interval of 100 years.

The reversible pump-turbine plant would serve two functions. Normally, the plant would provide about 300,000 kilowatts of hydroelectric power for the Chicago area, operating as a modern pump-storage project using the underground storage capacity as the lower reservoir. The same reversible units would operate as pumps to evacuate storm overflow waters from the underground reservoir.

The Plan would almost completely eliminate the risk of basement and underpass flooding in the project area and, as a result, would substantially enhance property values. It would lower the cost of new sewer construction in presently undeveloped areas. The Plan would make possible the treatment of all overflows from the combined sewer system, eliminating a source of pollution of the Illinois River waterway system and Lake Michigan. It would also reduce flood flows in the waterway system to the benefit of navigation and riparian property.

The initial stage project, in the Lake Calumet area, would be capable of extension to include the balance of the Chicago metropolitan area at lower unit costs through joint use of initial stage project facilities.

Justification for Feasibility Studies

We conclude that the Deep Tunnel Plan is highly promising. It appears likely it will cost no more, and possibly less, than alternative drainage and power production means. It offers a substantial bonus in the form of pollution control and flood reduction benefits which are not furnished by alternative proposals. It deserves investigation to the detail currently given to other alternatives.

We recommend that feasibility level investigations of the proposed plan be initiated. These studies not only would establish the engineering and economic feasibility of a project in the Chicagoland Area - they also would demonstrate the possibilities for other urban areas. Thus they could have national significance.

Scope of Services

The prefeasibility studies were carried out using existing and available geologic, topographic, and hydrologic data with a minimum of field exploration, survey, or testing. The purpose of the prefeasibility studies has been to determine on the basis of existing and available data if the project has sufficient merit to warrant feasibility level studies.

The feasibility studies, which are the subject of this proposal, consist of engineering studies and analyses carried out in sufficient detail to establish engineering, economic, and financial feasibility of the Deep Tunnel Plan. This phase would include drilling programs and surveys as necessary to support this demonstration of feasibility. We estimate that the feasibility studies will require 12 months to complete.

Our prefeasibility report describes the elements to be studied in the feasibility phase and the type and degree of analysis and exploration which would be carried out. These elements are summarized below.

a. Hydrologic studies which would include hourly studies of critical rainfall-runoff events; neighborhood drainage system characteristics; lateral and main sewer flow capacities and characteristics; studies and model tests of new main sewers, spillways, and vertical drains; hydraulic performance of the underground reservoir for drainage and power purposes; and determination of the effects of the Plan on flood stages and flows in the waterway system.

b. Sanitary engineering studies which would include study of potential health hazards associated with flooding of basements and streets and pollution of the waterways; determination of the quality of combined sewer overflows as related to quantity and duration; chemical and biological reactions in the sewers, vertical drains, underground reservoir, pumping-generating facilities, and the surface reservoir; potential quantity of trash and sediment inflow and methods for control and removal; requirements, methods, and facilities for treatment of overflow waters; and the extent and nature of pollution abatement in the receiving waterways attributable to the Plan.

c. Geological studies which would include further evaluation of existing data; preparation, administration, supervision, and interpretation of a drilling and testing program; and preparation of engineering criteria for the location and design of all physical components of the Plan.

d. Groundwater studies which would include further evaluation of existing well data; preparation, administration, supervision, and interpretation of an aquifer test program which would include a recharge test well, observation wells, and pump-test wells; preparation of a more detailed electric analog model of the aquifer system; and determination of water table migration, recharge requirements, and groundwater inflow quantity into the underground facilities.

e. Hydroelectric power studies which would include selection of optimum installed capacity, relationship of use of the underground reservoir for drainage and power based on hourly study of all critical runoff events, layout and preliminary design of the power facilities and machinery, and study of the value and cost of power generation.

f. Project layouts which would combine criteria and requirements of the above studies in optimum layouts of the physical components of the Plan toward the objectives of minimum cost and maximum utility.

g. Cost estimates which would include further substantiation of unit costs developed in the prefeasibility studies and determination of total project cost including the costs of new sewers, sewer modifications, vertical drains, underground excavation, pumping-generating facilities, surface reservoir, groundwater recharge, and water treatment.

h. Economic and financial studies which would include determination of tangible benefits attributable to reduction of basement flooding and other local flooding, reduction of flooding in the waterways, and hydroelectric power production; determination, description, and evaluation of intangible benefits; determination of secondary benefits and costs to the community attributable to the Plan; preparation of schedules and rates for construction and operation; and assistance in negotiations with financing institutions.

Organization

The Harza Engineering Company will maintain primary responsibility for all investigations. Bauer Engineering, Inc., which was responsible for the original idea of underground storage of drainage water, will participate in our studies.

The majority of the engineering work to be done in the feasibility studies will be carried out within the Harza and Bauer organizations. For problems that fall outside our range of experience, however, we customarily employ specialized consultants. We feel that such specialized consulting services would be desirable in the field of groundwater hydraulics, sanitary engineering, and possibly in the field of mining engineering.

For the groundwater aspects of the Deep Tunnel Plan, we propose to continue to use the services of Mr. William C. Walton, an independent groundwater consultant. Mr. Walton formerly was associated with the Illinois State Water Survey and is intimately familiar with groundwater conditions in the Chicago area. He would advise us in the general planning of the groundwater investigations, develop design criteria, review the detailed studies which we would make, and provide consulting advice during public or regulatory agency hearings.

We would continue to use the services of Dr. Bloodgood and Dr. Toebes of Purdue as sanitary engineering consultants to assist us in studies of the nature of the sewage introduced into the underground system, potential development of septic conditions in the underground and surface reservoirs, and the requirements for and type of sewage treatment needed as part of the system.

In the field of mining engineering, we would arrange for the services of appropriate consultants when and if particular needs should arise but do not, at this time, anticipate the need of broad services in this field.

Cost Estimate

We estimate that the engineering costs of the feasibility studies would be approximately $395,000, and the cost of drilling and groundwater testing programs would be approximately $382,000, or a total of $777,000. A summary of estimated cost distribution is presented below:

Engineering time	$313,000
Consultants and subcontracts	59,000
Direct costs	23,000
Total Engineering Cost	$395,000
Geologic and groundwater drilling and testing contracts	382,000
Total Cost	$777,000

The total cost is divided into categories of work as follows:

Item	Approximate Cost for Engineering Services	
HYDROLOGIC AND HYDRAULIC ASPECTS		
Engineering Studies	$109,000	
Rainfall and runoff		
Existing sewer system capacity survey		
Frequency of use		
Flood control and drainage		
Subcontracts - Model testing, mapping	43,000	
Subtotal		$152,000
POLLUTION ABATEMENT AND SANITARY ASPECTS		
Engineering Studies	34,000	
Relief of septic conditions		
Treatment of effluent		
Sediment quantity and removal		
Decrease in downstream pollution		
Subcontract - consultant	1,000	
Subtotal		35,000
GROUNDWATER ASPECTS		
Engineering Studies	30,000	
Supervision and data interpretation		
Analog computer analyses		
Subcontracts - Consultant, drilling and testing	285,000	
Subtotal		315,000
POWER ASPECTS		
Engineering Studies		
Energy and capacity potentials		
Joint use of facilities		
Power facilities layouts		
Subtotal		32,000
PROJECT LAYOUT AND CONSTRUCTION		
Engineering Studies	86,000	
Component sizing and layouts		
Alternative project layouts		
Supervision of drilling		
Geologic interpretation		
Subcontracts - Drilling and testing	107,000	
Subtotal		193,000

Item	Approximate Cost for Engineering Services
ECONOMIC ANALYSES	
Engineering Studies	
Project costs	
Pollution control benefits	
Drainage and flood control benefits	
Benefit-cost relationships	
Subtotal	35,000
PREPARATION AND PUBLICATION OF FEASIBILITY REPORT	15,000
TOTAL	$777,000

We propose to be reimbursed under the same terms as our present contract with the Sanitary District for the prefeasibility studies of the Deep Tunnel Plan. The schedule of charges would be in accordance with Form 3200-A, a copy of which is attached. The services of Bauer Engineering, Inc. would be reimbursed under the same terms.

Services of William C. Walton, Dr. Bloodgood, Dr. Toebes, and other consultants would be under subcontracts and would be reimbursable as direct expenses.

Contracts for geological and groundwater exploration programs would be made on the basis of competitive bids and contractors would be selected by Harza Engineering Company with the approval of the Sanitary District. Exploration contracts would be administered by Harza and would be reimbursable as a direct expense.

We are grateful for this opportunity to submit this proposal and will be pleased to discuss it at your convenience.

Very truly yours,

BAUER ENGINEERING, INC.

W. J. Bauer

W. J. Bauer
President

HARZA ENGINEERING COMPANY

Arthur P. Geuss

Arthur P. Geuss
Executive Vice President

Enc: Form 3200 - A

Form 3200-A

HARZA ENGINEERING COMPANY
CONSULTING ENGINEERS
RIVER PROJECTS
400 WEST MADISON STREET
CABLE ADDRESS "HARZENG CHICAGO" CHICAGO 6, ILLINOIS TELEPHONE RANDOLPH 6-3451

SCHEDULE OF CHARGES

A. Compensation

For the consulting engineering services covered by the accompanying proposal letter, Harza Engineering Company shall be paid monthly the sum of

1. Direct Salaries, plus
2. Overhead Costs and Fee, computed at 100% of Direct Salaries, plus
3. Direct Costs

B. Definitions

1. Direct Salaries - Actual Salary payments to all personnel, including officers, for time directly engaged on the work. The maximum salary rate to be charged for any person will be $12.50 per hour.

2. (a) Overhead Costs - Costs which cannot be allocated to specific projects such as vacation time, sick leave time, holidays, idle time between assignments, general stenographic and clerical payroll, rent of office and drafting room space, utilities, depreciation of office equipment, etc.

   (b) Fee - Payment to the Company for costs not otherwise reimbursed, for availability of the engineering organization and facilities, for the accumulated knowledge and experience of the firm, and for profit to the Company.

3. Direct Costs - Costs which are directly applicable to the Project such as transportation and subsistence expenses on travel in the interest of the Project, long distance telephone, telegraph and cable expenses, reproductions, model and laboratory testing, subsurface exploration, electronic computer time, etc.

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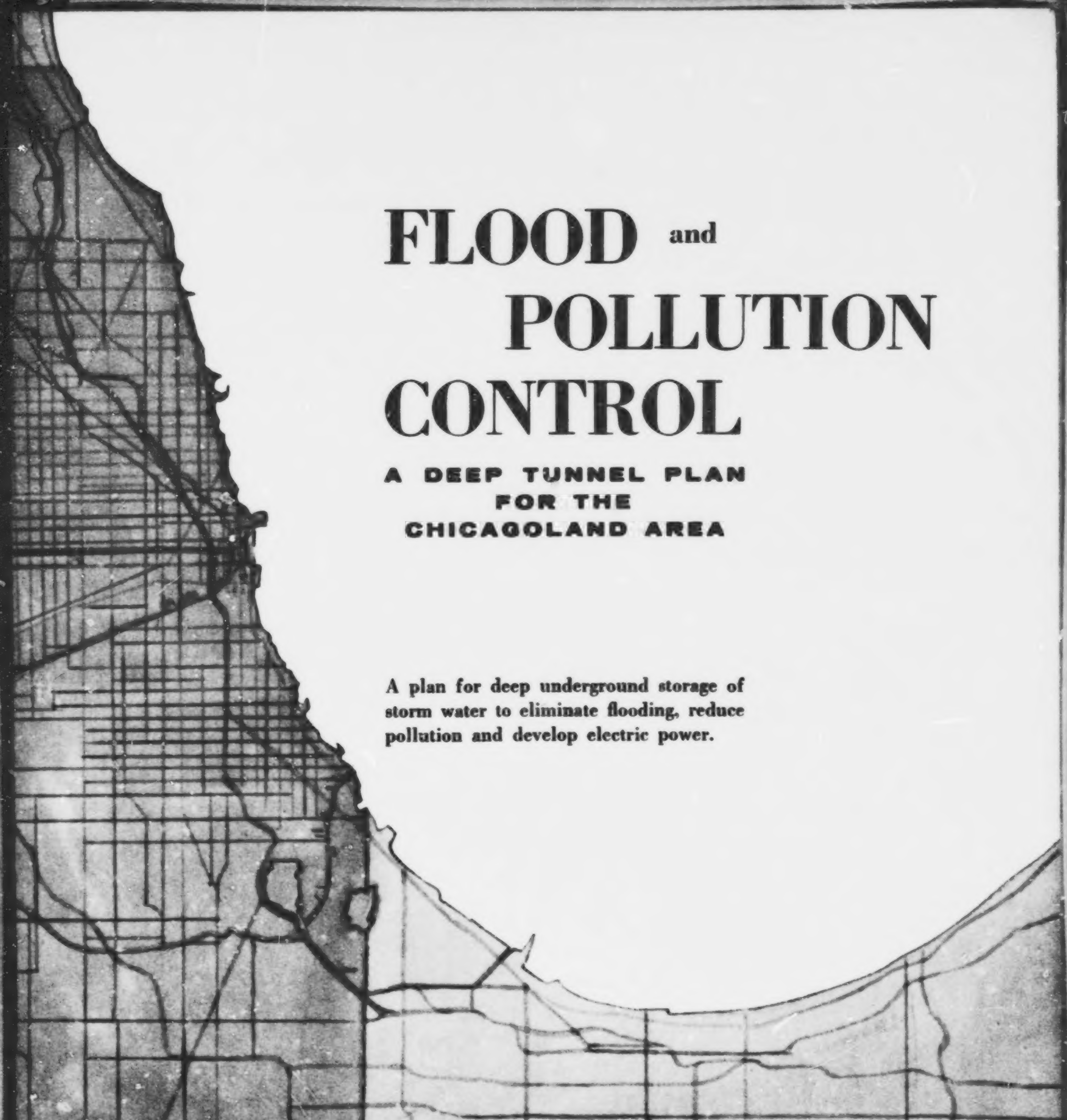


FLOOD and POLLUTION CONTROL

A DEEP TUNNEL PLAN FOR THE CHICAGOLAND AREA

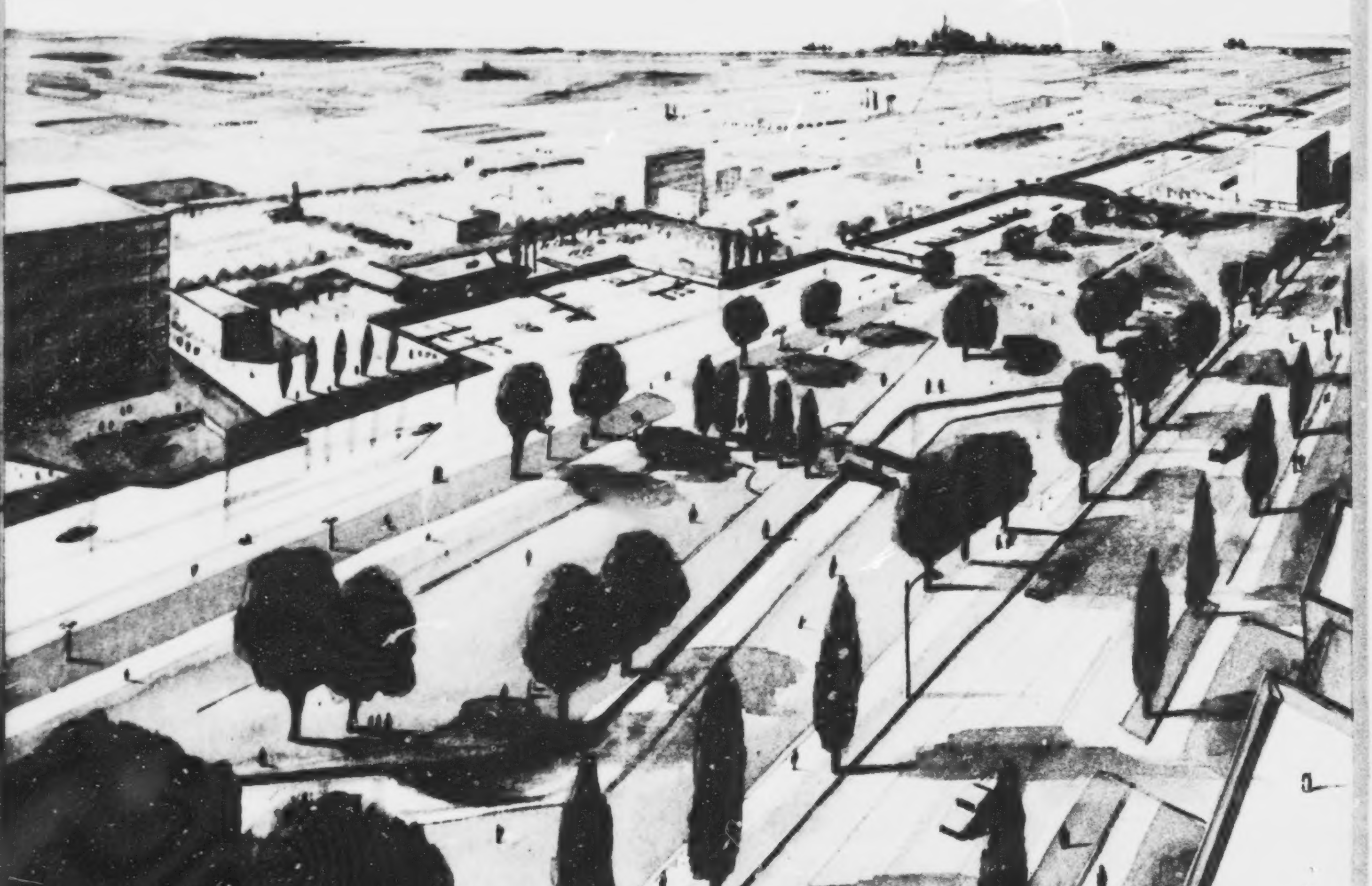
A plan for deep underground storage of storm water to eliminate flooding, reduce pollution and develop electric power.

THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO

May 1966

Clean Waterways and Flood-Free Lands

This booklet describes a completely new concept being explored by the Sanitary District in its program of reducing stream pollution and flood damage in the Chicagoland Area.



Deep Underground Storage of Chicagoland Area Storm Water

A totally new concept, called the Deep Tunnel Plan for Flood and Pollution Control, is being explored by the Sanitary District. The plan envisions temporary storage underground of the contaminated storm water originating from overloading of sewers. This storage would occur in a system of tunnels and large chambers excavated in solid rock, deep under the City. After the end of a storm, the stored water would be pumped to the surface where it would be treated to remove pollution before being discharged into the waterways.

The system would practically eliminate basement and underpass flooding. If expanded to the entire Chicagoland Area, it would eliminate polluted overflows into the rivers and channels of the area, as well as to Lake Michigan. It also would be used for generation of hydroelectric "pumped-storage" power.

The District and its consulting engineers have completed an initial engineering study of the plan. The Deep Tunnel Plan opens new potentials for solution of a vital problem affecting not only Chicago but other urban areas. A pioneering solution of this problem by Chicago would be in keeping with the engineering leadership traditions of the Sanitary District.

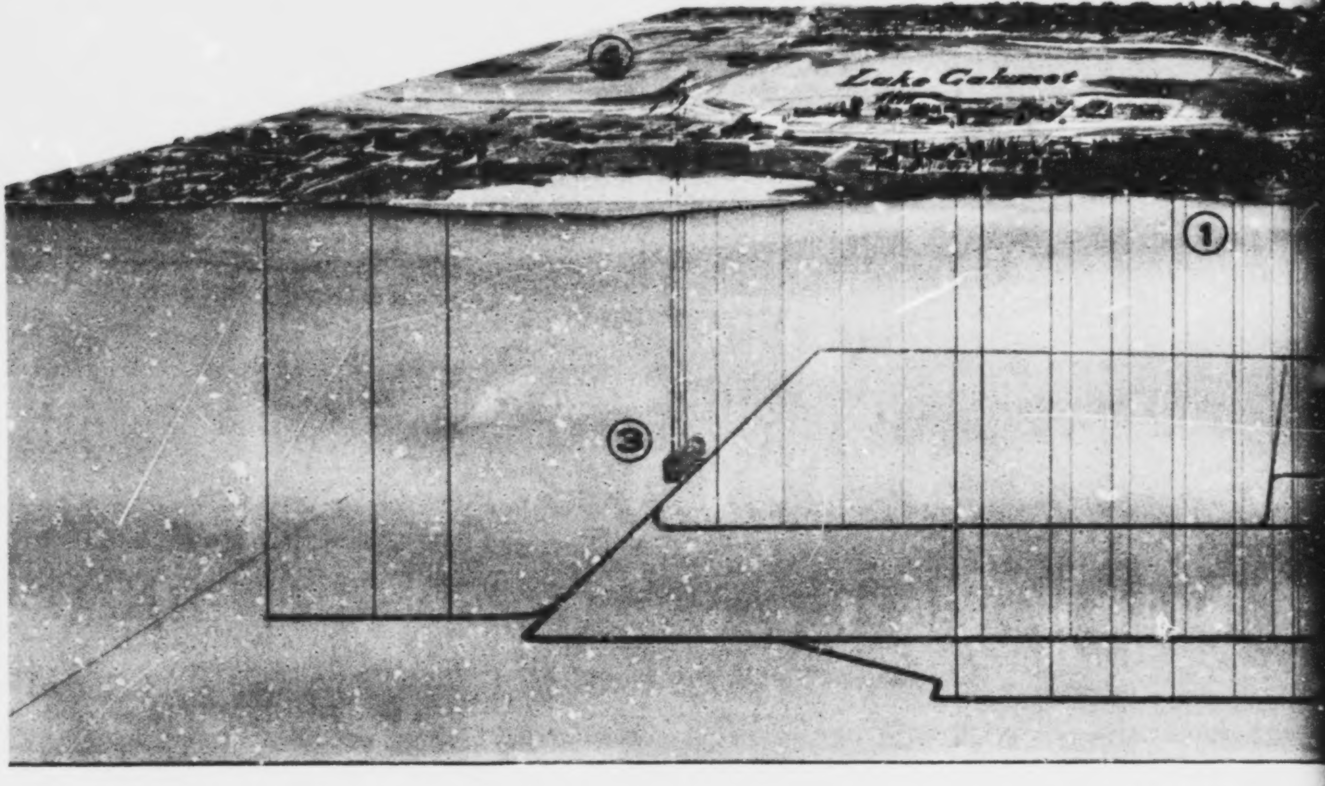
The Metropolitan Sanitary District of Greater Chicago

Frank W. Chesrow
President

Vinton W. Bacon
General Superintendent

Board of Trustees
John B. Brandt *Frank W. Chesrow* *Earl A. Deutsch* *John E. Egan* *Vincent D. Garrity*
Valentine Janicki *Francis P. Kane* *Nicholas J. Melas* *Earl E. Strayhorn*

This project was supported in part by Demonstration Grant No. WPD 106-01-66 from the Research and Training Grant Program, Federal Water Pollution Control Administration



How t

THE PROBLEM TODAY

Storm flows and domestic sewage are carried by the same sewers in much of the Chicagoland Area. During heavy rainstorms, these sewers overflow, carrying rainwater mixed with raw sewage directly into the area's rivers and channels, and under severe conditions into Lake Michigan. These combined-sewer overflows, along with industrial pollution and heavier loads on sewage treatment plants, have virtually eliminated all recreational use of the waterways and destroyed fish life in much of the Illinois River system.

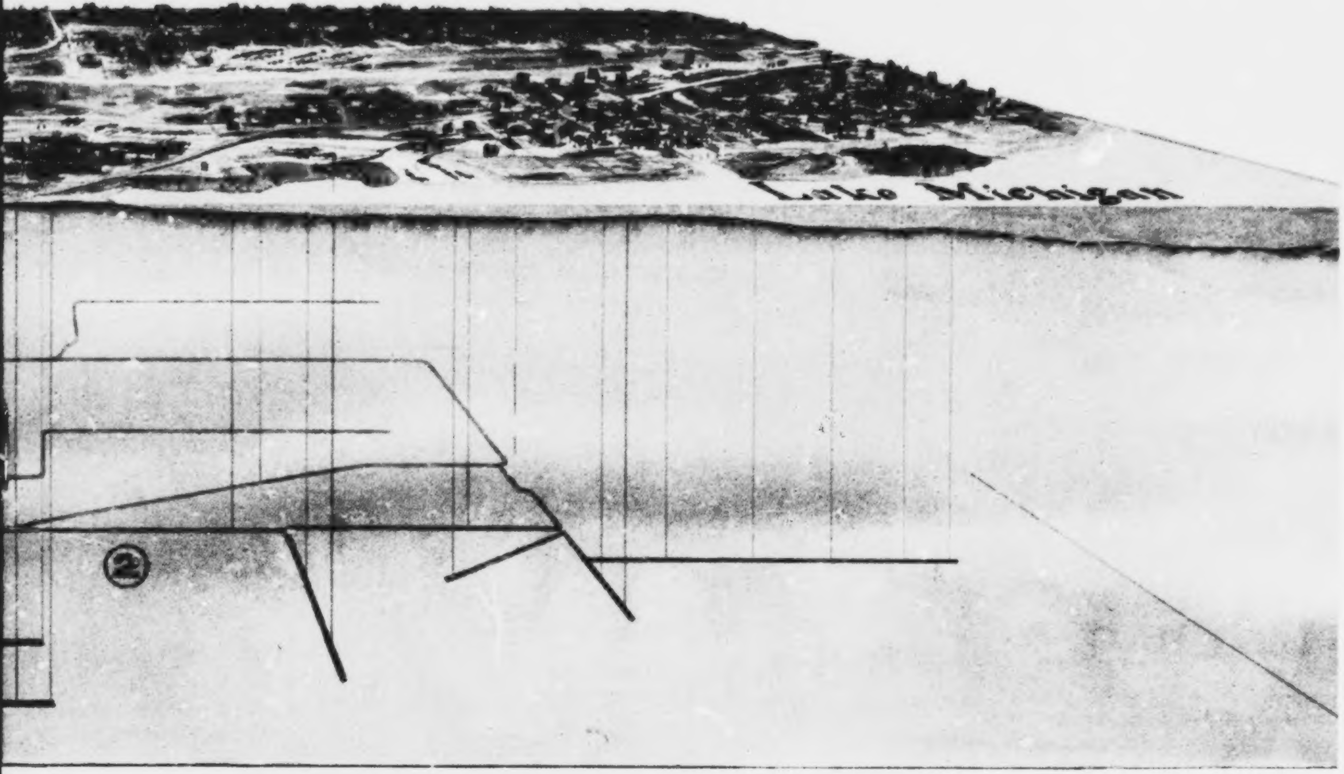
Because of the area's very flat topography, the lateral and main sewers in many cases cannot protect completely against basement and underpass flooding. A widely used solution for flooding problems is to store flood waters in reservoirs, to be slowly and harmlessly released. A flood storage reservoir would also allow time for treatment of the pollution in the overflow waters. This solution, however, is difficult to use in completely urbanized areas. More often, basements themselves become reservoirs. Recognizing the impracticality of building surface reservoirs in the urban area, the idea of building reservoirs underground was conceived.

①

Excessive rain and snow-melt water, mixed with domestic sewage, will be withdrawn from the existing sewer network through vertical drains at strategic locations.

Removing excess flows from points within the sewer network will have the effect of increasing sewer capacity without replacing the sewers. It will also stop overflows into the rivers and channels and Lake Michigan.

The vertical drains will range from 12 to 72 inches in diameter. The vertical drains also serve to admit air to the underground reservoir.



e Deep Tunnel Plan Will Operate

②

The vertical drains will discharge contaminated storm water into an underground reservoir, made up of a network of tunnels and chambers excavated in solid rock 700 to 800 feet below the land surface.

The location of the underground reservoir has been selected to make use of a rock formation that will not require structural bracing for support. The underground reservoir will both store water and convey it to a single removal point.

Special precautions will be taken to protect the groundwater from contamination.

③

A pumping station, located in a chamber in the rock, will remove the water stored in the underground reservoir at a convenient time and rate of flow.

The pumps and their electric motors will be reversible so that they can also operate as turbines and generators.

In addition to the removal of storm water, the reversible pump-turbine station will operate as a "pumped-storage" hydroelectric plant, similar to plants already being built near other large cities. Water would be dropped from the surface during the day to generate high-value peaking power and pumped back up at night using low-cost off-peak power.

④

After being pumped from the underground reservoir, water would be temporarily stored in a reservoir on the surface before being discharged into the waterways.

This reservoir, located on unused land, would store water for power generation. Pollution would be removed from the overflow water either by aeration in the reservoir or by transfer of the contaminated water to the Calumet Sewage Treatment Plant.

The circulation and aeration involved in power generation would contribute greatly to the purification process.

HARZA ENGINEERING COMPANY
CONSULTING ENGINEERS
RIVER PROJECTS
400 W. MADISON STREET, CHICAGO, ILL. 60606

BAUER ENGINEERING, INC.
CONSULTING ENGINEERS
ENGINEERING HYDRAULICS
20 N. WACKER DRIVE, CHICAGO, ILL. 60606

May 16, 1966

The Metropolitan Sanitary District
of Greater Chicago
100 E. Erie Street
Chicago, Illinois 60611

Gentlemen:

We are pleased to report the completion of prefeasibility engineering studies for the Deep Tunnel Plan for Flood and Pollution Control which you asked us to undertake on November 1, 1965. Our studies show that the concept of relieving overloaded combined sewers by storage of surplus waters in an underground reservoir, excavated in sound rock formations, has great promise.

We have studied records and data obtained by State agencies and other institutions to determine the suitability of the rock, the quantity of overflow water to be stored, and the degree of pollution of the overflow water. Because of its great influence on the potential feasibility of the concept we also have studied the methods and cost of deep rock excavation in greater detail than normal for this level of investigation.

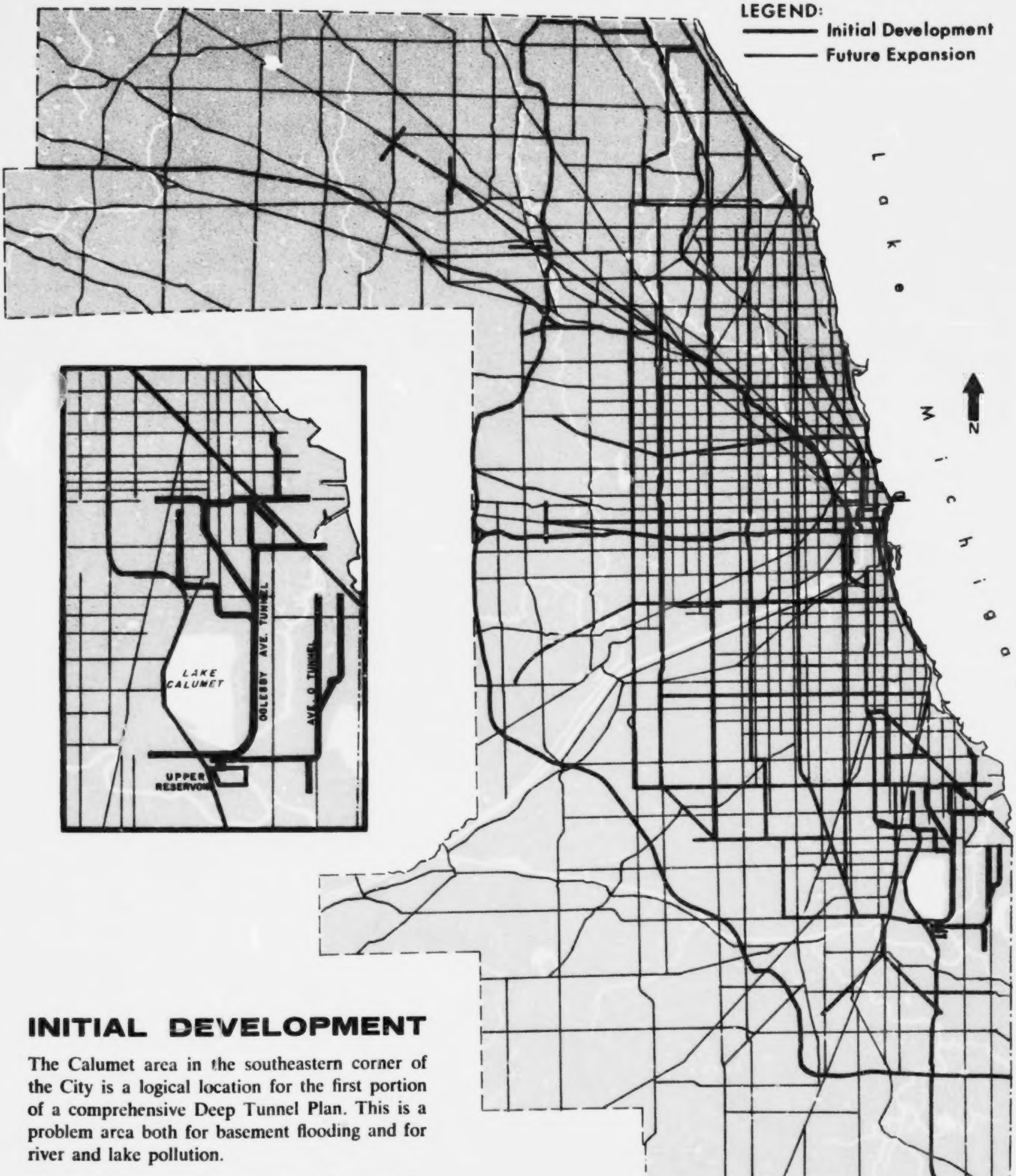
We conclude that construction of an underground flood storage reservoir in solid rock should present no major difficulty. We find that the Plan would be more effective and less costly than alternative methods of pollution abatement and flood control.

As a next step, we recommend that feasibility level studies and geological explorations, which will be necessary for project financing, be carried out for an initial development in the area near Lake Calumet. An initial development in this location could be readily extended to serve the remainder of the area under the jurisdiction of the Metropolitan Sanitary District.

Very truly yours,

HARZA ENGINEERING COMPANY
Richard D. Harza
Richard D. Harza
Vice-President

BAUER ENGINEERING, INC.
W. J. Bauer
William J. Bauer
President


LEGEND:
Initial Development
Future Expansion
Lake Michigan
N
LAKE CALUMET
OGLESBY AVE. TUNNEL
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INITIAL DEVELOPMENT

The Calumet area in the southeastern corner of the City is a logical location for the first portion of a comprehensive Deep Tunnel Plan. This is a problem area both for basement flooding and for river and lake pollution.

The Calumet area is also ideally suited engineering-wise for ultimate expansion to an area-wide system.

HARZA ENGINEERING COMPANY
CONSULTING ENGINEERS
RIVER PROJECTS
400 W. MADISON STREET. CHICAGO ILL. 60606

BAUER ENGINEERING, INC.
CONSULTING ENGINEERS
ENGINEERING HYDRAULICS
20 N. WACKER DRIVE. CHICAGO. ILL. 60606

May 13, 1966

Mr. Vinton W. Bacon,
General Superintendent
The Metropolitan Sanitary District
of Greater Chicago
100 East Erie Street
Chicago, Illinois 60611

Dear Mr. Bacon:

We are pleased to submit our report on prefeasibility studies of a Deep Tunnel Plan for flood and pollution control in the Chicagoland Area. This project was supported, in part, by Demonstration Grant No. WPD 106-01-66 from the Research and Training Grant Program, Federal Water Pollution Control Agency.

Your authorization for these studies was based on our proposal that we appraise the general feasibility of the Deep Tunnel Plan utilizing only existing geologic, groundwater and other data. Under this approach, costly geologic borings and geologic tests, together with detailed engineering studies as necessary to support full engineering and economic feasibility, would await the results of this appraisal.

The purpose of this report, therefore, has been to (a) appraise the performance and general feasibility of the Plan, (b) compare its costs and benefits with other alternatives and (c) determine whether it offers sufficient promise to justify detailed engineering investigations.

The general conclusion of our studies is that basic objectives of flood and pollution control can be achieved by the proposed plan at reasonable cost. We further conclude that detailed engineering feasibility studies are warranted and we recommend that they be carried out.

The Flood and Pollution Problem

The Chicagoland Area is experiencing major problems of flood and pollution control. Pollution of the area's waterways from combined sewer overflows and other sources is increasing, while rainstorm flooding of basements and roadways remains a recurrent problem. These problems stem from the continued economic growth and urbanization of the area, as well as from the inherent limitation of the existing combined sewer system. The present nationwide interest in higher levels of flood protection and water quality serves to emphasize the need for an even higher degree of flood and pollution control in the Chicagoland Area.

Under present conditions, rainstorms of even moderate intensity cause overloading of the combined sewers. Industrial waste discharges, disposal of raw sewage by individuals, and the residual from even high-quality sewage treatment plant effluent, combine with storm water overflow as sources of pollution in the waterways. The rivers now fall short of the minimum requirements for any sort of water recreation and cannot support any but the poorest fish life.

While each of the other pollution sources is under strong attack, no practicable means has been found as yet to control the combined sewer overflows. At present such overflows represent 16 percent of the pollution load of the waterways, but after other sources of pollution are reduced this presently unsolved problem may become dominant.

The conventional combined-sewer system also frequently fails to perform its primary function of protecting residences, industry, and commerce from destructive flooding. The most common result of limited sewer capacity is basement flooding, which, in addition to actual damage, restricts the uses to which basement space can be put. Flood damages also occur in the waterways and these would be increased if conventional combined-sewer capacities were increased.

It has long been recognized that an effective solution could be obtained for the flood and pollution problems if space could be provided for temporary storage of storm-water overflows. Such temporary storage would allow time for treatment of the water to reduce pollution and would make possible the controlled release of overflow water to the waterway system. However, because of the flat terrain, surface sites for flood storage reservoirs are not available in the urban area at the locations needed for flood protection.

Description of the Deep Tunnel Plan

The proposed Deep Tunnel Plan offers a solution to the problems of flood and pollution by providing a deep underground reservoir, excavated in solid rock, in place of the more usual surface reservoir. Although the concept represents an entirely new approach to control of floods and pollution, the proposed system is made up of conventional structures, equipment, and construction methods.

General Arrangement

Flood and pollution control for the Chicagoland Area can be accomplished in convenient stages. The proposed initial stage, shown on Exhibit A, is for a 21-square mile area near Lake Calumet. The general concept of the plan is shown on Exhibit B. The key elements in operation of the plan are as follows:

(a) Interception of storm-water overflows from existing or newly constructed main sewers by vertical drains. Existing laterals would be incorporated into the plan.

(b) Discharge from the vertical drains into a 2820 acre-foot underground reservoir, excavated in solid rock approximately 800 feet below the surface.

(c) After temporary storage underground, pumping of the storm-water overflows to a surface reservoir constructed south of Lake Calumet, having a storage capacity of 2360 acre-feet.

(d) Utilization of the upper and lower reservoirs for hydroelectric pumped-storage operation, by adding generating facilities to the required pumping equipment.

(e) Treatment of the polluted storm-water overflows by either (1) settling, screening and aeration available in the normal operation of the pumped-storage cycle or (2) utilization of surplus capacity of the Calumet Sewage Treatment Plant.

(f) Discharge of treated effluent to the waterway system at controlled rates.

The area to be protected is relatively flat. Drainage has been a recurring problem, with marsh or open water in low-lying areas. Conventional sanitary sewers and storm drainage have not been fully developed for the area. As a result, the land development is extremely diverse, as follows:

Residential and commercial	4,880 acres
Industrial, railroads, and highways	3,700
Public land	1,570
Vacant land	2,700
Water surface	720
Future Lake Calumet fill	220
Total	13,790 acres

The plan provides an extremely high level of flood protection, by incorporating a drainage design for a flood of 100-year frequency. This objective is attained for substantially all types of storms. The capacities of existing laterals are greatly increased by affording relief at both the upper and lower ends of the laterals. The necessity for such relief accounts for a large proportion of the proposed new main sewers.

Underground Reservoir

The proposed underground reservoir would consist primarily of a network of tunnels in sound rock. The tunnels would be 33 feet in diameter, 23.4 miles in length, and would be distributed throughout the drainage area. The purpose of the tunnels would be to provide storage, as well as to convey water to a central point for pumping to the surface.

It is probable that the tunnels would be constructed by a tunneling machine (called a "mole"). Extensive investigation of construction methods and costs indicates that this method of construction is both practical and economical for tunnels. Bulk excavation at a centralized location can be accomplished more economically by "drill-and-blast" methods, and this method is proposed for about one-eighth of the required storage volume.

The storage capacity of the system will be large enough to handle the runoff from the largest storm in Chicago's 94 years of record. However, the underground reservoir will be empty of storm water most of the time, which will allow joint use of about 40 percent of its volume as a lower reservoir for a revenue-producing hydroelectric pump-storage plant.

Hydroelectric Power

Hydroelectric pumped-storage plants do not represent a new concept. They have been utilized for more than 60 years in Europe and 12 large pumped-storage plants are either in existence or under construction in the United States. Pumped-storage plants have become increasingly important in future planning of the larger utilities, because of the advent of reversible pump-turbine units and the ever-increasing difference between on-peak and off-peak power loads.

The proposed pumped-storage plant will have a pumping capacity of 3750 cubic feet per second. Normally it will pump water from the lower reservoir to the upper reservoir during night time hours, when surplus generating capability exists in the power system. Release of water in the reverse direction during peak load hours will add a generating capability of 300,000 kilowatts at a time when the remainder of the capability of the power system is normally taxed to near its capacity.

The addition of hydroelectric pumped-storage facilities is not a necessity for operation of the project. However, it is attractive economically. With pumping capacity being otherwise necessary, the incremental cost of adding reversible units is relatively small. An upper reservoir is required, but it also serves a dual purpose, replacing some of the underground storage for drainage purposes. Discussions with the Commonwealth Edison Company indicate that pumped-storage facility would greatly aid their operations. Final decisions by that company would depend on a number of factors, including the allocation of the cost of power, which has not been determined at this time.

Water Quality Improvement

The daily circulation of storm water in the pumped-storage cycle will produce an inherent form of sewage treatment. The turbulence in the storm water falling into the lower reservoir will reduce the size of solids. Settling of solids will take place in the lower reservoir, except for finer entrained material pumped to the upper reservoir. Air drawn into the lower reservoir will be favorable to oxygen transfer and the surface reservoir of about 135 acres will function as an oxidation pond. Baffling, mechanical aeration in the surface reservoir, or introduction of air at the pump-turbines could significantly enhance the treatment performance, if needed.

While we feel confident that these "inherent" treatment features of the plan will be entirely adequate, the opportunity exists to pass all of the storm-water overflow through the Calumet Treatment Plant. If necessary, surplus treatment capacity is available at the plant to accomplish this. The cost of a line from the upper reservoir to the Treatment Plant has been included in the overall cost estimate of the plan.

Aquifer Protection

Special attention has been paid to the problem of protecting the existing aquifers in the area. Our studies indicate that adequate and economical protection can be provided. We conclude that the use of a system of recharge wells is the most likely method and have included the cost of such wells as a part of the plan.

Cost of Deep Tunnel Plan

Our preliminary estimate of the construction cost of the Deep Tunnel Plan is summarized as follows:

New main sewers	$11,700,000
Vertical drains	3,400,000
Underground reservoir	33,200,000
Pump-turbine station	25,700,000
Ventilation	1,200,000
Surface reservoir	5,800,000
Pipeline to Calumet Plant	1,000,000
Aquifer recharge	3,200,000
Total	$85,200,000

The dominant construction cost factor of the plan is the cost of underground excavation. We have given this matter careful study and have considered machinery costs, Chicago labor rates and rules, and construction activity scheduling. Excavation unit prices were calculated to be $5.50 per cubic yard for underground quarrying and $6.00 per cubic yard for the tunneling machine. These unit costs were reviewed by independent experts.

Justification of Deep Tunnel Plan

The principal benefits of the Deep Tunnel Plan fall into three categories: pollution control, urban drainage, and power production. Of these, only power production is readily susceptible to economic evaluation. Calculation of monetary benefits from pollution control and urban drainage is not a usual requirement for justification of such projects. Any benefit-cost analysis that may ultimately be possible for the Deep Tunnel Plan is reserved for the later feasibility stage. For purposes of this report, a comparison with alternative costs is deemed sufficient.

The City of Chicago is currently investigating the cost and feasibility of a "Conventional Plan," which would provide conventional combined sewers for the same 21-square mile drainage proposed for the Deep Tunnel Plan. Up-dated cost estimates are not completed, but estimates prepared in 1960 indicated the probable cost of conventional sewers to be $62,000,000. That estimate furnishes a basis for comparison with that of the Deep Tunnel Plan.

The hydroelectric features of the Deep Tunnel Plan are not essential to its operation. They are included because they are revenue-producing and add to the economic feasibility of the plan. Therefore, the best comparison of costs between alternative drainage plans can be obtained by deducting alternative costs of power from the cost of the Deep Tunnel Plan.

Because of the preliminary stage of project evaluation, the cost of alternative peaking capacity for the hydroelectric features of the project has been evaluated on the basis of conceivable minimum and maximum values. This range is estimated to be $70 to $110 per kilowatt of installed generating capacity. On this basis, the alternative cost of a 300,000-kilowatt plant would be $21,000,000 to $33,000,000. Subtracting these values from the total estimated cost of about $85,000,000 leaves a cost of between $52,000,000 and $67,000,000 for providing pollution and flood control in the Deep Tunnel Plan

The cost of $52,000,000 to $67,000,000 is of the same order of magnitude as the $62,000,000 estimate made in 1960 for the Conventional Plan. Since the Deep Tunnel Plan affords much greater benefits, the comparison in costs indicates the Deep Tunnel Plan to be highly promising. The additional benefits attributable to the Deep Tunnel Plan include the following:

(a) The Deep Tunnel Plan will allow storage and treatment of storm-water overflows from combined sewers, which is not included in the Conventional Plan, as now conceived.

(b) The Deep Tunnel Plan will provide flood protection and drainage for a design objective of a 100-year flood, as compared with a 5-year flood in the Conventional Plan, and it will provide relief to more existing sewers than those to be relieved by the Conventional Plan.

(c) The Deep Tunnel Plan will provide a potential for tertiary treatment of sewage plant effluent and for spare capacity for treatment of some dry-weather flow.

(d) The Deep Tunnel Plan will add operating flexibility and efficiency to the Chicago sewage treatment and storm-flow system because of the vastly enlarged storage available underground.

Conclusion and Recommendations

We conclude that the Deep Tunnel Plan is highly promising. It appears likely it will cost no more, and possibly less, than alternative drainage and power production means. It offers a substantial bonus in the form of pollution control and flood reduction benefits which are not furnished by alternative proposals. It deserves investigation to the detail currently given to other alternatives.

We recommend that feasibility level investigations of the proposed plan be initiated. These studies not only would establish the engineering and economic feasibility of a project in the Chicagoland Area - they also would demonstrate the possibilities for other urban areas. Thus they could have national significance.

The recommended program of feasibility studies is outlined in detail in the Report. They would consist of both office and field investigations. About half of the cost of the feasibility studies would be in the field, consisting of borings to explore the geologic structure in the Calumet area and of groundwater tests in the area.

It has been a pleasure to serve on this most interesting investigation of a new concept for pollution and flood control in the Chicago area. We will be pleased to answer any questions you may have.

Yours very truly,

BAUER ENGINEERING INC.

W. J. Bauer

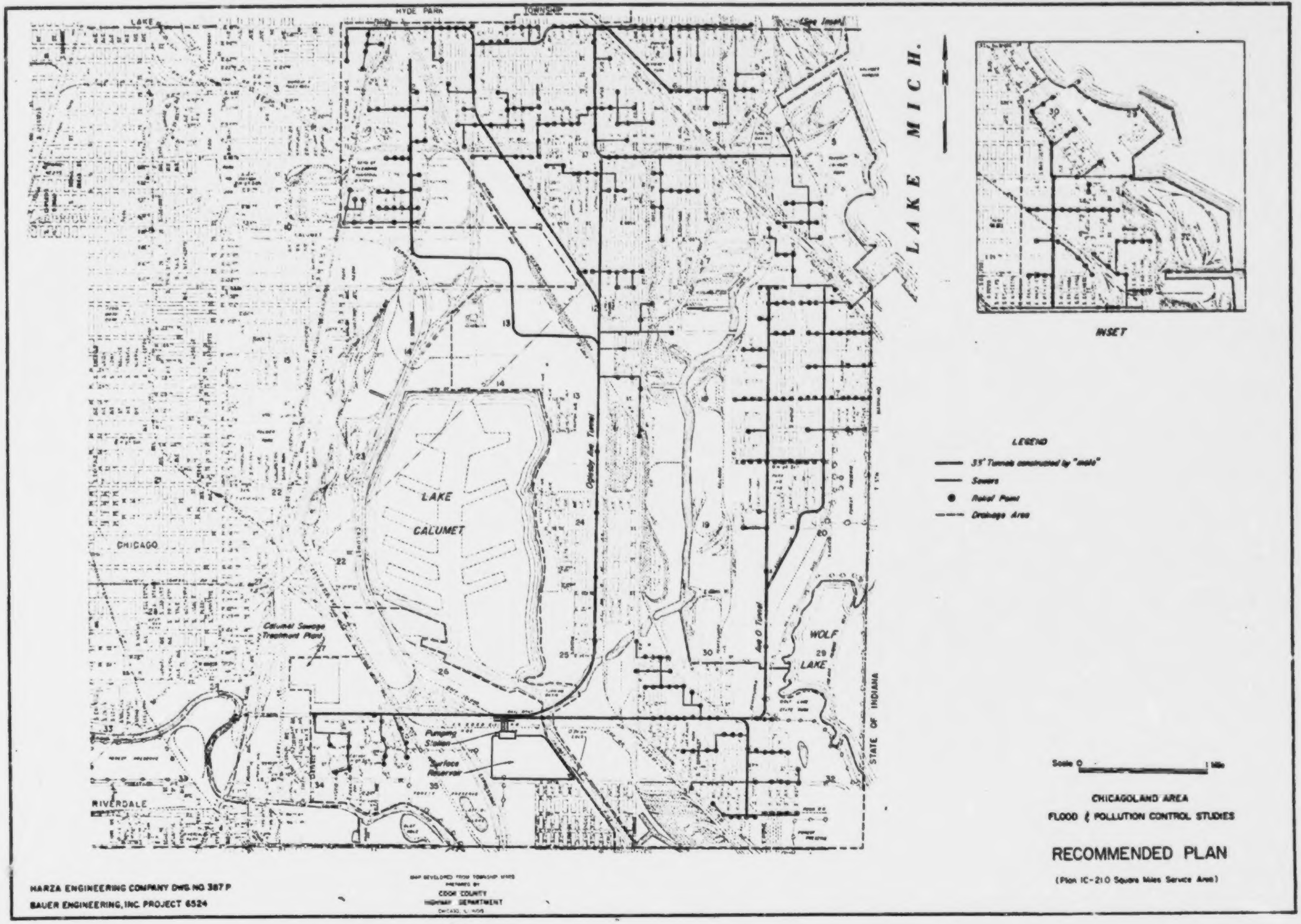
W. J. Bauer
President

HARZA ENGINEERING COMPANY

Richard D. Harza

Richard D. Harza
Vice President

EXHIBIT A



LAKE MICH.
HYDE PARK TOWNSHIP
(See Inset)
LAKE
CALUMET
CHICAGO
Ogelsby Ave Tunnel
Ave O Tunnel
WOLF LAKE
STATE OF INDIANA
Calumet Sewage Treatment Plant
Pumping Station
Surface Reservoir
RIVERDALE
INSET
Scale 0 1 Mile

LEGEND

- 35' Tunnels constructed by "mole"
- Sewers
- ● Relief Point
- Drainage Area

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

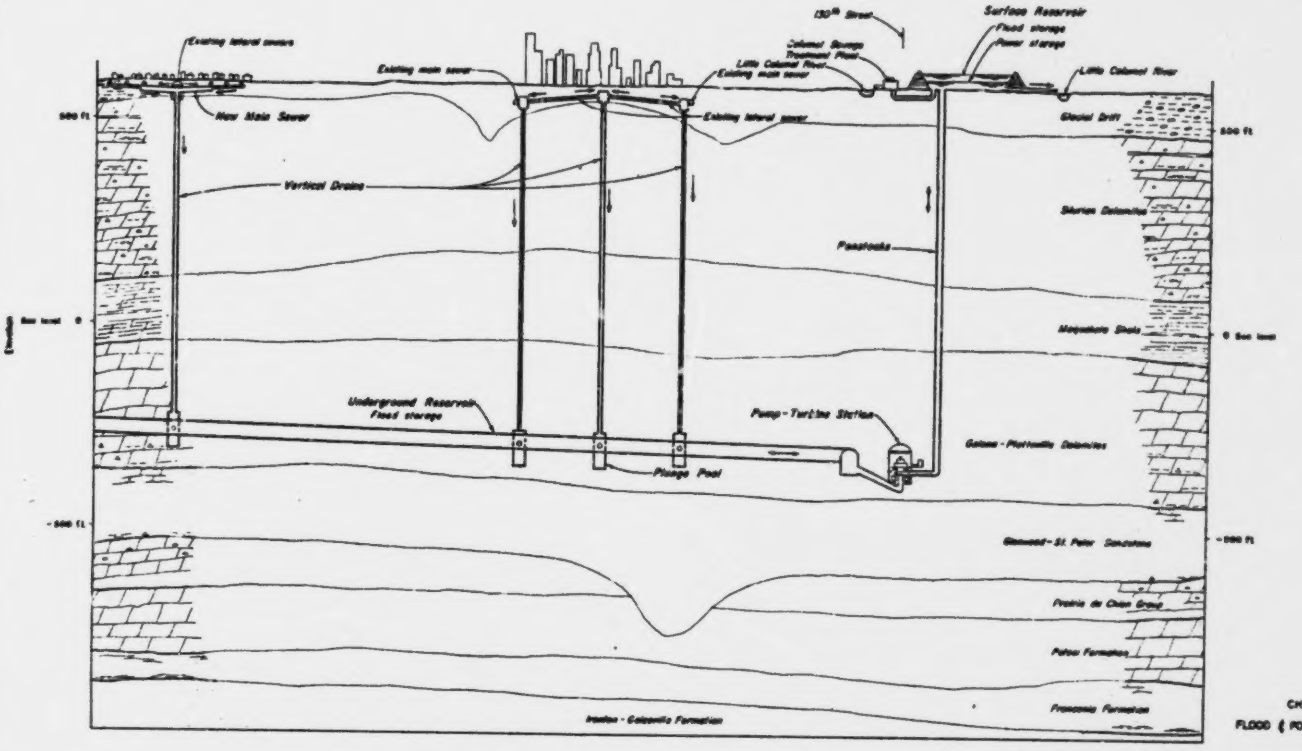
RECOMMENDED PLAN

(Plan IC-210 Square Miles Service Area)

HARZA ENGINEERING COMPANY DWG NO 387 P
BAUER ENGINEERING, INC. PROJECT 6524

Map developed from township maps
prepared by
COOK COUNTY
HIGHWAY DEPARTMENT
CHICAGO, ILLINOIS

EXHIBIT B



Existing lateral sewers
New Main Sewer
Vertical Drains
Existing main sewer
Little Calumet River
Existing main sewer
Calumet Sewage Treatment Plant
130th Street
Surface Reservoir
Flood storage
Power storage
Little Calumet River
Existing lateral sewer
Glacial Drift
500 ft
Silurian Dolomites
Penstocks
Elevation
Sea level 0
Maquoketa Shale
0 Sea level
Underground Reservoir
Flood storage
Pump-Turbine Station
Galena-Platteville Dolomites
Plunge Pool
-500 ft
Glenwood-St. Peter Sandstone
Prairie du Chien Group
Potosi Formation
Franconia Formation
Ironton-Galesville Formation

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

GENERAL CONCEPT

HARZA ENGINEERING COMPANY DWG.NO. 387 P-1
BAUER ENGINEERING, INC. PROJECT 6524

Abbreviated Table of Equivalents

Water Supply Equivalents

1 cubic foot per second	equals	448.83 gallons per minute
1 cubic foot per second	equals	1.98 acre feet/day
1 million gallons per day	equals	1.55 cubic feet per second
1 acre foot	equals	326,000 gallons
1 inch runoff/hour	equals	1.01 cfs/acre
1 inch runoff	equals	53.3 acre-feet/sq. mi.

Abbreviations

Acre-foot	AF
Biochemical Oxygen Demand	BOD
Cubic feet per second	cfs
Dissolved Oxygen	DO
Dry-weather flow	DWF
Gallons per day	gpd
Kilowatt	kw
Liter	l
Megawatt	MW
Million gallons per day	mgd
Parts per million	ppm
Population equivalent	PE
Sewage Treatment Works	STW

Definitions of Terms

Aquifer	water-bearing formation which will yield usable quantities of water to wells penetrating it.
Biochemical Oxygen Demand	the amount of oxygen required to completely decompose sewage or waste water pollutants; normally measured by laboratories as five-day BOD with mathematical conversion to ultimate or total BOD.
Coliform organism	members of a parasitic bacteria group prevalent in the intestinal tract of warm-blooded animals; survive for long periods in water, easily detected by laboratory methods; thus a good indicator of pollution by human waste.
Combined sewers	sewers functioning concurrently as storm water drains and as sanitary sewers.
Conventional sewers	those sewers at shallow depth (i. e., 5 to 25 feet below surface) normally constructed by open cut-and-fill methods.
Drainage area	the land area which contributes surface rain or snowmelt runoff to a stream or sewer system.
Load	the amount of electric power delivered at a given point.
Load factor	the ratio of the average load over a designated period to the peak load occurring in that period.
Outage	the period during which a generating unit, transmission line, or other facility, is out of service.
Peak load	the maximum load in a stated period of time.

Present worth	the monetary amount today which would exactly repay a future payment or series of payments at a given interest rate.
Runoff	the water from rainfall or snowmelt which flows off of the land into streams or sewers.
Pumped-storage station	a power plant arranged so that electric energy is generated for peak load use by utilizing water pumped into a storage reservoir - usually during off-peak periods. A pumped-storage station may also be used to provide reserve generating capacity.
Service area	the land area which contributes drainage to a sewer system.
Thermal power station	a term used to identify a type of electric generating station or power plant in which the source of energy for the prime mover is heat (whether nuclear or fossil fuels are used).

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III. Engineering Design Criteria

IV. The Recommended Plan

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VII. Recommended Investigation Program

Chapter I

INTRODUCTION

The Problem

Although it is an acknowledged leader in the field of storm drainage and sewage treatment, growing Chicagoland is still experiencing major problems of storm and sanitary water management. Pollution of the metropolitan area's waterways by storm-sewer overflows is increasing while rainstorm flooding of basements and roadways remains a recurrent problem. Both of these problems stem from inadequacies in the existing combined sewer system. Expansion of this sewer system along conventional lines does not promise permanent relief.

Waterway Pollution

Rainstorms of even moderate intensity cause overloading of the sewers and overflows of storm water - usually mixed with raw sewage - into the Chicago, Des Plaines, and Calumet River systems and, under severe conditions, into Lake Michigan. Although much of the land along the rivers and lake has been preserved and developed for public recreation and enjoyment, the waters themselves are increasingly desecrated with sewage. Public policy is swinging strongly in favor of restoring and preserving the quality of these waters for recreation, fishing, and esthetic enjoyment.

Gross pollution exists in the waters of the Upper Illinois River System, shown on Exhibit 1, as noted by both casual observation and scientific studies. Rubbish, garbage, oil slicks, and floating sewage solids, along with gas bubbles from decomposition of sludge deposits, may be observed along most of the waterway above Lockport. Field investigations and laboratory tests have shown consistently high levels of less visible forms of pollution.

The Illinois River Waterway has undergone tremendous change in the last 100 years. Population in the watershed has grown from about a half million in 1850 to over 8.5 million in 1960. Diversion of Lake Michigan water and Chicago sewage started through the Illinois-Michigan Canal in 1871. Diversions from the Lake Michigan were greatly increased in 1900 by opening of the Sanitary and ship canal with beneficial results on pollution conditions. About this same time, however, river regime was severely upset by agricultural drainage of flood plains. Diversions from the lake were drastically reduced from 1935 to 1940 by decree of the U. S. Supreme Court, but, despite these reductions, concurrent treatment improvements by the District greatly improved conditions in the Waterway. Since 1940, the accelerated growth of Chicago and associated waste output has caused another increase in pollution, despite additional remedial measures.

Several parameters, shown graphically on Exhibit 2, demonstrate the extent of this deterioration. The reported turbidity of river water has increased two-to three-fold in the past 20 years due to the extensive development in the watershed of row crop agriculture, barge-tow operations, and sewage pollution. A dissolved oxygen level of four parts per million is considered a minimum for sport fish, but it is now often near zero throughout the waterway. As a result, 18 species of fish which once were supported no longer survive. Aquatic vegetation and lower animal life have largely disappeared in the upper reaches of the river system, resulting in drastic reductions in migratory water fowl and fish populations.

In addition to deterioration of the waterway, even the beaches of Lake Michigan are restricted for swimming at times of major overflows of storm drainage to the Lake. Although coliform counts of 5000 are considered a limit for boating, the counts in the waterway reach 1,000,000 at times. River pollution not only affects the Chicago area, but extends the full length of the Illinois River.

Storm overflows from combined sewers in the Metropolitan Sanitary District service area contribute a significant amount of this pollution. About three to five percent of the total domestic sewage is passed untreated into the waterways along with storm overflows, while up to 95 percent of domestic sewage in the sewer system can be carried into the waterways during the overflow period of major storms.

Combined sewer overflows from the Metropolitan Sanitary District presently contribute about 16 percent of the total BOD (biochemical oxygen demand) loading at Lockport. The Public Health Service estimates that this proportion will become 24 percent after other scheduled improvements are carried out. Overflows presently contribute an average of 56,000 pounds of ultimate BOD per day, of which 4,500 pounds are estimated to be contributed by the Calumet area.

Other sources combine with sewer overflows to pollute the rivers. These include disposal of raw sewage by individuals and small communities, poor quality sewage treatment plant effluent, and industrial waste discharges. Each of these other sources is under strong attack. Sewage treatment is being extended and improved, and industrial polluters are under orders to treat their wastes or find alternative means of disposal. However, for lack of feasible methods, no action is contemplated against pollution caused by sewer overflows.

Table I-1 summarizes present BOD loadings and the estimated result of improvements taken from the "Report on the Illinois River System - January 1963" prepared by the U. S. Public Health Service.

Table I-1

BOD LOADING - UPPER ILLINOIS RIVER

Upper Illinois River System - Lake Michigan to Lockport

Present Total Load in pounds of ultimate BOD per day

a.	Municipal	226,000
b.	Industrial	71,000
c.	Calculated Storm Spillage	56,000
	Total Load	353,000

BOD Reduction by Improvements

a.	Secondary Treatment of Municipal Waste	10,000
b.	Industrial Waste Reduction	58,000
c.	Industrial Waste Ordinance - MSD	10,000
d.	Chlorination of MSD Effluents	38,000
e.	Stormwater Overflow Control through Short-Term Improvements	6,000
	Total Pounds of BOD Reduction by Improvements	122,000
	Total BOD Load After Improvements	231,000

The possibility of separating the storm and sanitary sewers has been considered as a means of reducing pollution by storm overflows. However, this could only be carried out at staggering costs, estimated as high as $17,000 per acre. This would amount to over two billion dollars for the entire City of Chicago. Research is being pressed to find more economical ways of sewer separation, but even total separation of sanitary sewers from storm sewers will not eliminate pollution. Recent U. S. Public Health Service studies in Cincinnati showed that storm runoff from separate sewers in urban areas is itself significantly contaminated. On the average, urban storm runoff could be said to have about the same quality as effluent from a mediocre sewage treatment plant. Storm water runoff from urban areas must itself be treated if pollution of the receiving waterways is to be minimized.

Detailed descriptions of the waterway pollution problem, its effects, and measures considered for pollution abatement are presented in Appendix G.

Rainstorm Flooding

Not only does the conventional sewer system fail to protect the waterways from pollution, it often fails to perform its primary function of protecting residences, industry, and commerce from destructive flooding. Because of flat topography in the Chicagoland Area, sewer flow capacity is very expensive, requiring large-size sewer pipes because of flat grades and pump-lifts at many outlets. High costs associated with these requirements make it prohibitive to design conventional sewers for storms greater than would be expected about once in five years.

The most common result of limited sewer drainage capacity is basement flooding. This flooding damages property and utilities in the basement, and often the structure itself, as well as presenting a potential health hazard. Perhaps of equal importance, the ever-present threat of flooding limits the uses which could be made of basement space. Many basements might be developed for living or recreation space if the flood threat were eliminated. Other damages caused by rainfalls which overtax sewer capacities are detours resulting from underpass flooding and damage to commercial and industrial property.

Overflows from the sewer system into the waterways often cause flooding of low lands adjoining the waterways, and may cause surges which interfere with navigation. Presumably, if drainage capacities of the conventional sewers were increased, downstream waterway flooding would become even more severe. Surface reservoir sites for temporary detention of storm runoff to minimize waterway flooding and to allow for treatment of overflows are not commonly available at suitable locations.

Further descriptions of flooding and estimates of monetary losses caused by floods are given in Chapter VI and Appendix G.

Present Investigations

New sewer developments in the City of Chicago generally follow along the lines of the existing system and fall short of a lasting solution for the pollution and flooding problems. In 1964, Bauer Engineering Inc. and Harza Engineering Company jointly suggested a new method for attacking these problems. This method would make use of large excavated tunnels and chambers, located deep underground, to both store and transport storm water overflows.

The proposed scheme has been designated as the Deep Tunnel Plan for Flood & Pollution Control. In its simplest concept, it provides the underground reservoir as an underground replacement for the more common surface flood control reservoir, combined with a high capacity interceptor sewer system. Exhibit 3 presents the concept schematically, and Exhibit 4 is a definition sketch for present and proposed system components.

After careful consideration of the possibilities of the Deep Tunnel Plan, the Sanitary District obtained demonstration grant funds from the U. S. Public Health Service for study of the plan and, on November 1, 1965, assigned the two companies to jointly carry out appraisal studies. Prior to that time, the Sanitary District had asked the Bureau of Engineering of the City of Chicago to make a study of flood relief in the Calumet Area of the City by means of conventional sewers. (In the context of this report conventional sewers are described as sewers near the ground surface, flowing by gravity to the treatment plants, except for occasional booster stations. Such sewers are usually laid in open cuts or tunnels in clay.)

The existence of the City study afforded an opportunity for direct comparison of conventional sewer methods with the Deep Tunnel Plan, both in performance and in costs. The Sanitary District directed the companies to study a deep tunnel plan using the same geographic boundaries. One case to be studied called for use of the same hydrologic design criteria as used for the conventional system - five-year storm protection and no treatment of overflows. The second case was to use the far more demanding criterion of 100-year storm protection and treatment of essentially all sewer overflows.

Because of the unique concepts of the Deep Tunnel Plan, it was considered desirable to obtain preliminary indications of project feasibility before going into an extensive exploratory drilling program. Therefore, the studies in this report are based almost entirely upon published data or data and information from other agencies, supplemented only by field reconnaissance. The investigations of the Deep Tunnel Plan reported here are, therefore, a first phase, or "prefeasibility level" engineering analysis.

The Calumet Area

The Calumet Area comprises nearly 35 square miles of widely diverse industrial, residential land, and undeveloped lands lying in the extreme southeastern portion of the City of Chicago adjacent to Lake Calumet. The area is generally bounded on the east by Lake Michigan and the Indiana Line; on the south by 134th Street; on the west by Halsted Street; and on the north by 87th Street.

The land surface is relatively flat except for low ridges along the western and northern fringes of the area. Drainage has been a perennial problem because of the low-lying, flat terrain and the absence of improved drainage. Ground water levels are commonly only a few feet below land surface, and marsh or open water prevails in some low-lying areas.

Sanitary sewers and storm drainage have not been developed in the Calumet Area to the level found in most parts of Chicago. Storm and sanitary sewer overflows occur along the Calumet River System even in those portions of the Area that have been sewered. Most of these overflows will be kept out of Lake Michigan by the recently completed O'Brien Lock and Dam, but they will continue to flow to the Illinois River Waterway.

Land development in the area is extremely diverse as shown by Table I-2 which gives land classification for the gross area to be served by the proposed conventional sewer plan for the Calumet Area.

Table I-2

CALUMET AREA
LAND CLASSIFICATION

Land Use	Conventional Plan Service Area	Total Calumet Area
Residential & Commercial	4,880 acres	7,380 acres
Indust, Railway, Highway	3,700	5,500
Public Land (connected to sewers)	770	1,120
Vacant Land(sewerable)	2,700	3,700
Future Lake Calumet Fill	220	450
Sub-total	12,270	18,150
Public Land (not connected to sewers)	800	1,550
Water Surface*	720	1,670
Totals	13,790 acres = 21.5 sq. mi.	21,370 acres = 33.4 sq. mi.

Future development of the relatively large amount of open land would be enhanced by improved drainage because of the inherent reduction of flood hazards and the availability of highly developed transportation facilities in the area.

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* not including surface reservoir

Chapter II

THE UNDERGROUND STORAGE CONCEPT - A PROPOSED SOLUTION

Overview of the Concept

The underground reservoir concept is an entirely new and original approach to controlling floods and pollution in the Chicago area. No other single system will produce the same results, and this complicates comparison with alternative conventional methods of flood and pollution management. Yet, despite the originality of the concept, the underground storage approach comprises only conventional time-proven structures and machines.

Both flooding and waterway pollution could be eliminated if (a) it were possible to remove storm water from the land surface as rapidly as it would accumulate, and (b) if overflow water could be stored long enough to allow all of the water to be treated and released at controlled rates. This could be accomplished by a reservoir or reservoirs large enough to contain all overflows and located close enough to all flooding areas so that inflow travel times would be short.

The Chicago area is very flat, with little topographic relief. This situation, combined with nearly complete development of the urban area, precludes use of surface reservoirs to accomplish the above objectives inside the urban area. This led to the development of the concept of storing sewer overflows in a system of large, deep excavated tunnels and chambers, located directly under present and future sewer relief points.

The proposed underground reservoir system would receive lateral and main sewer overflows through vertical shafts. The underground reservoir would drain by gravity to an evacuation point from which the polluted water could be pumped to the surface, then given some form of treatment, if desired, before disposal into the waterway system.

Since the entire capacity of the underground reservoir would be used only infrequently for flood storage, it became apparent that the normally empty underground space could serve as the lower reservoir of a pumped-storage power plant. The location and configuration of the underground facilities present an unusual opportunity for such an installation which would, in turn, help to pay for the flood and pollution control facilities. Inclusion of a pumped-storage power plant would require the addition of an upper reservoir at the land surface to store water required for peak power generation. The power plant would serve the electrical power system in a variety of ways. The water dropped for generation during daytime peak-load hours would be pumped back up to the surface reservoir the following night when power loads are low and surplus generating capacity is available.

The addition of a surface reservoir makes possible still another function of the system. Preliminary studies indicate that the surface reservoir also could serve as very large oxidation pond. Other parts of the system function to accomplish trash removal, sedimentation, agitation, and aeration in much the same manner as do components of conventional sewage treatment plants. These studies indicate that the system may have a very large potential capacity as a system for sewage treatment, possibly much larger than the capacity merely to treat storm water overflows. Thus, there is a real prospect that the system may be operated to perform sewage treatment on a large scale, in addition to serving the needs of storm drainage, flood control, and power generation.

A signigicant feature of the proposed system is that it could be constructed by stages to serve ultimately the entire City and much of the

Chicagoland area. The proposed first stage of the system would include the essential facilities for that stage only, but these facilities would be located and designed to allow increases in capacity as additional stages expanded the service area. The construction of the first stage would in no way imply a commitment for future expansion.

Alternative Plans

The City of Chicago is currently investigating conventional sewerage improvements in the Lake Calumet area, affording a basis for comparison with the underground storage approach. The improvements being studied by the City would serve the area generally bounded by 138th Street, State Street, 87th Street, Lake Michigan, and the Indiana State Line. General boundaries and drainage units of the Calumet area are shown on Exhibit 5. The total contributing drainage area north of 138th Street would be approximately 13,500 acres (21.0 sq. mi.) although the improvements contemplated by the City would affect an area of about 19,000 acres (29.7 sq. mi.) north of 138th Street, not including water surfaces and forest preserve areas within the project boundaries.

The project area is attractive for the location of the first stage of an underground storage plan since it includes the Calumet Sewage Treatment Works, water transportation facilities for excavation disposal, and adequate open space for a surface reservoir which could be expanded to serve the entire metropolitan area. The major geologic formations which lie under the City generally slope downward toward the project area, making it a logical evacuation point for a gravity underground drainage system.

Several alternative layout studies were made to study the underground storage concept which are referred to throughout the remainder of this report as "Deep Tunnel" plans. The principal alternative layouts were selected to develop a comparison of functions, costs, and benefits of the underground concept with the City plan for conventional sewerage improvements. Other alternatives were selected to portray different service areas and alternative construction methods. Except as noted, the bulk of the excavation in the "Deep Tunnel" plans would be by tunneling machine or "mole." The alternatives are as follows:

(a) Conventional Plan. The City of Chicago conventional sewerage plan serving 21.0 square miles.

(b) Deep Tunnel Plan 1A. An underground storage plan with the same functions, design criteria, and service area as the City Plan.

(c) Deep Tunnel Plan 1B. A plan identical in scope to Plan 1A except for the addition of pumped-storage power, The surface reservoir included in this plan for power would also allow for treatment of all storm overflows.

(d) Deep Tunnel Plan 1C. A plan identical in scope to Plan 1B except for addition of new main sewers and additional relief points. This plan would provide flood protection against a 100-year storm.

(e) Deep Tunnel Plan 2A. A plan identical in concept to Plan 1C except for enlargement of the service area to 29.7 square miles.

(f) Deep Tunnel Plan 2B. A plan identical in scope to Plan 2A, with the exception that all underground excavation would be carried out by conventional drilling and blasting.

Details of the alternative plans are summarized in Table II-1 and are discussed in the following sections.

Conventional Plan (21.0 sq. mi.)

The City of Chicago Conventional Plan is shown on Exhibit 6. The key element of this plan is a new 6000 cubic feet per second pumping station

Table II-1

COMPARISON OF ALTERNATIVE LAYOUTS

Layout Designation	Service Area (sq. mi.)[1]	Miles of Sewers	Miles of Tunnels	Volume of Underground Reservoir	Power Capability MW
Conventional Plan	21.0	26.9	----	-----------	---
Plan 1A	21.0	0.6	25.2	3.6 inches[2] 4050 acre-ft	---
Plan 1B	21.0	0.6	25.2	2.5 inches 2810 acre-ft	300
Plan 1C	21.0	29.8	23.4	2.5 inches 2810 acre-ft	300
Plan 2A	29.7	53.4	38.6	2.5 inches 3960 acre-ft	400
Plan 2B	29.7	69.5	13.0	2.5 inches 3960 acre-ft	400

1/ Area served does not include forest preserves, Lake Calumet, and other non-sewerable areas.

2/ Inches depth over the service area.

located on the east side of the Calumet River at 134th Street. This station would receive drainage from about 22.6 square miles, of which about 18.0 square miles lie north of 138th Street. The exact size of the area served depends upon the hydrological conditions during each storm, as the direction of flow in some main sewers varies with the location of centers of rainfall intensity. The average drainage area was assumed to be 21.0 square miles. The 134th Street Pumping Station would be capable of discharging into the Calumet River either north or south of O'Brien Locks.

Both dry-weather and storm flows would be conveyed to the 134th Street Pumping Station by 26.9 miles of new interceptor sewers. The largest of these, the Oglesby Avenue Interceptor system, would drain the area presently served by the 95th Street Pumping Station (which would be abandoned) and the central and northern portions of the project area. The Brandon-Burley Interceptor system would drain the eastern portion of the project area, and the 134th Street (west) Interceptor system, the southwestern portion. In addition to the 95th Street Pumping Station, the Hegewisch, Altgeld Garden, and possibly, the 122nd Street Pumping Stations would be abandoned.

The interceptor sewers would relieve existing main sewers and lateral sewers along their routes. Some relief would be afforded to the area west of Lake Calumet by transfer of much of the load of the existing South Park Interceptor to the new Oglesby Avenue Interceptor. Dry-weather flow would be pumped to the Calumet Sewage Treatment Works through a new main sewer from the 134th Street Pumping Station.

The Conventional Plan is designed to handle a five-year storm with a duration of three hours or longer. The Plan appears to be capable of evacuating water from the service area at the rate of approximately three-eighths inch per hour. No treatment is contemplated in this plan for the combined-sewer storm flows, which would be pumped into the Calumet River.

Plan 1A - Limited Deep Tunnel Plan (21.0 sq. mi.)

Plan 1A represents a limited function Deep Tunnel plan that would exactly parallel the Conventional Plan in function, service area, and in hydrologic design criteria. This plan was prepared for cost comparisons only. It is not proposed as an actual alternative to more comprehensive underground plans described in the next sections.

Plan 1A consists of a system of 25.2 miles of 33-foot diameter tunnels which would exactly follow the alignment of the interceptor sewers in the Conventional Plan. Main and lateral sewers would be relieved at the same locations as in the Conventional Plan, by means of vertical drains connecting to an underground reservoir system comprised of the tunnels and excavated chambers. An underground pumping station with a capacity of 500 cubic feet per second would be located similarly to the 134th Street Pumping Station in the Conventional Plan. Less pumping capacity is required in Plan 1A because of the detention storage available in the underground reservoir.

The total underground storage in Plan 1A would be 4050 acre-feet, equivalent to 3.6 inches runoff from the service area. This storage volume, which is much greater than the 0.8 inch runoff associated with the five-year design storm provided for in the Conventional Plan, would be necessary so that the two plans would perform in the same manner during storms greater than the five-year design storm. Thus, the degree of residual neighborhood flooding resulting from storms larger than the five-year design storm would be the same with both plans. The selected volume of 3.6 inches is based on the total amount of water which would have flowed through the Conventional Plan sewers during the most critical historical storm, that of October 1954.

It was assumed in Plan 1A that dry-weather flow would not be dropped into the underground system but would continue to be delivered to the Calumet Treatment Works by the existing main sewers, interceptors, and pumping stations. The larger storm pumping facilities at the existing pumping stations would be abandoned as in the Conventional Plan. A low-flow sewer of 150 cubic feet per second capacity would connect the underground pumping station with the Calumet Treatment Works so that a portion of the polluted flood overflows temporarily stored in the underground system could be treated before disposal in the Calumet River.

Plan 1A was designed to be exactly comparable to the Conventional Plan in its benefits from reduction of neighborhood flooding. However, as a result of matching the neighborhood flood reduction benefits in the Conventional Plan, other important benefits would accrue to Plan 1A, without additional cost. These benefits are in flood reduction and pollution abatement in the receiving waterway system. Because of the moderate pumping rate requirement in Plan 1A, the pumping station discharges would not cause flooding along the waterways nor would it ever be necessary to divert polluted water to Lake Michigan as would have to be done under severe conditions with the Conventional Plan. Also, the overflows associated with frequent minor storms could be passed through the Calumet Treatment Works, by "off-peak" pumping of overflows stored in the underground reservoir. The overflows from these minor storms have the highest pollution concentrations because of less dilution by rain water.

Plan 1B - Limited Deep Tunnel Plan Plus Power (21.0 sq. mi.)

Plan 1B would serve the same area and meet the same hydrologic criteria as would Plan 1A and the Conventional Plan. It differs from Plan 1A in that it includes a 300,000 kilowatt pumped-storage hydroelectric station. Plan 1B also includes a surface reservoir capable of storing 2360 acre-feet (2.1 inches depth over the drainage area). Of this amount, 1130 acre-feet would be required for power use, leaving 1230 acre-feet normally empty for storage of stormwater overflows. The very large pumping capacity available for pumped-storage operation (3750 cfs) makes possible the storage of part of the stormwater overflows in the more economical upper reservoir. This reduces the underground storage requirement from 3.6 inches to 2.5 inches, or 2810 acre-feet.

Plan 1B would have the same benefits as the Conventional Plan, plus the potential of treatment of storm water overflows by the surface reservoir and possibly by aeration during pumping and generating, as well as the value of the power generation capacity.

Plan 1C - Comprehensive Deep Tunnel Plan (21.0 sq. mi.)

Plan 1C, shown on Exhibit 7, is the recommended plan. It is designed to derive maximum benefits attributable to the unique features of the underground storage concept. This plan differs from Plan 1B by addition of 29.8 miles of new main sewers to relieve existing main and lateral sewers. The new sewers would convey excessive flows from the existing sewers, through vertical drains, to the underground reservoir. The service area of Plan 1C is the same as that of the Conventional Plan, 21.0 square miles, in order that relative merits may be compared. The addition of new main sewers allows a change in tunnel alignment and slight reduction in tunnel length from Plans 1A and 1B.

Plan 1C would be capable of eliminating virtually all flooding within the project area, except for rare minor flooding caused by temporary overloading (runoff in excess of 1.6 inches per hour) of the relieved lateral sewers. The Plan would be capable of handling runoff from the largest storms that occurred during the 94 years of record in Chicago, without significant basement flooding and without overloading the waterway system. Plan 1C would make possible the treatment of essentially all of the combined sewer overflows which presently contribute to the pollution of the receiving waterways and , in addition, probably have spare treatment capacity for other uses. This Plan would include a pumped-storage hydroelectric station to increase beneficial utilization of the system's components.

Plan 1C would achieve great improvement in the abatement of neighborhood flooding over that of the Conventional Plan or Deep Tunnel Plan 1B. New main sewers would be provided to relieve all existing main sewers and to increase the capacity of existing lateral sewers by relieving their upper ends. Basement flooding due to inadequate capacity of existing lateral sewers would be virtually eliminated. (Existing sewer capacity would be increased on the order of ten times without enlargement)

Plan 1C is described in greater detail in the following chapter - "Recommended Plan."

Plan 2A - Comprehensive Deep Tunnel Plan (29.7 sq. mi.)

Plan 2A, shown on Exhibit 8, would provide the same comprehensive functions and benefits as Plan 1C, but the service area would be enlarged to 29.7 square miles, 8.7 square miles larger than the Conventional Plan. This enlargement was studied to determine the effect of size on overall unit costs. About 38 miles of tunnels, 33 feet in diameter would be arranged to serve both presently developed and presently vacant land. About 55 miles of new sewers would be constructed as part of this layout. This layout was designed on the assumption that all excavation of the underground reservoir would be done by tunneling machine or "mole."

Plan 2A would have a total underground storage of 3960 acre-feet, equal to 2.5 inches depth of the 19,000 acres of contributing drainage area. Water in the underground reservoir would be evacuated by reversible pump turbines with 400,000 kilwatts total capacity located in an underground station. Water pumped from the underground reservoir would be stored in a 3320 acre-foot surface reservoir.

Plan 2B - Alternative Comprehensive Deep Tunnel Plan (29.7 sq. mi.)

Plan 2B would be similar in function and service area to Plan 2A, with components of the plan modified to be more suitable to tunnel excavation carried out by traditional drilling and blasting methods. In order to achieve maximum economy with this method of construction, it would be desireable to concentrate the storage volume near the main access shaft and to limit the tunnel size to that required for hydraulic conveyance of design storm flow. Seventy miles of additional main sewers would be required in this plan.

Selection of Recommended Plan

Layouts and cost estimates were prepared for each of the alternative plans described above. The estimated total costs for each of the alternative plans are summarized in Table II-2. The methods used in preparing these estimates and the costs of individual plan components are described in Chapter V and Appendix F.

Table II-1

ALTERNATIVE PLANS COST SUMMARY

Conventional Plan	- 21.0 sq. mi.		$ 62,000,000
Plan 1A	- 21.0 sq. mi.	- limited function	64,600,000
Plan 1B	- 21.0 sq. mi.	- plus power	76,000,000
Plan 1C	- 21.0 sq. mi.	- comprehensive	85,200,000
Plan 2A	- 29.7 sq. mi.	- comprehensive (mole)	118,800,000
Plan 2B	- 29.7 sq. mi.	- comprehensive (drill & blast)	135,800,000

Alternative Plan 1C was selected as the recommended plan because:

(a) There is no unit cost advantage to the larger size project since the cost per square mile of the comprehensive project for the two sizes are virtually the same; and, therefore,

(b) The lowest cost alternative comprehensive plan should be selected since it would be a smaller first step and more readily financible.

The comparison between Plan 2A and Plan 2B indicates that construction by "mole" would be the most economical.

The recommended plan is described in detail in Chapter IV.

Chapter III

ENGINEERING DESIGN CRITERIA

Certain design criteria for the Deep Tunnel Plan are considered to be mandatory or desirable. These criteria are concerned with the functional capability, operation, and safety of the project.

Hydrologic Criteria

The basic hydrologic criterion used in sizing the system components for purposes of the prefeasibility study was that all new facilities should have capacity for a 100-year recurrence runoff event. In addition, all existing main sewers should be relieved so as to increase their capacity to two inches per hour which is adequate to handle recurrences of all historical storms without backing up. Capacity of lateral sewers should be increased to the maximum degree possible by relieving their upper and lower ends. Lateral sewers would than accommodate 100-year rainfalls of longer than 100 minutes duration.

The basic criteria used for determination of runoff rates and volumes are those developed by the Bureau of Engineering of the City of Chicago and are described in Appendix A.

Overflow Treatment

It was considered essential that all combined sewer overflows would be treated prior to discharge into the Calumet River, so that the effluent would at least meet the effluent quality standards maintained by other sewage treatment plants discharging into the river. Provision was made in the cost estimates in this study for transporting storm-water overflows to the Calumet Sewage Treatment Works. This allowance has been made for purposes of conservatism, even though the routine operation of the Deep Tunnel system will constitute inherent sewage treatment which will probably achieve the required effluent quality standards.

Aquifer Protection

It was assumed that the project must incorporate positive protection to domestic water supply aquifers from contamination by the polluted water stored in the underground reservoir. It was concluded, and confirmed by the State Board of Health and the State Water Survey, that satisfactory protection could be obtained only by maintaining the groundwater piezometric level (pressure water table) higher than the underground reservoir. With groundwater always at higher pressure than the water inside the reservoir, all seepage would be into the reservoir. In this manner contaminated water could not escape. Tunnel linings or any other barriers to seepage outflow were judged inadequate because of difficulties in inspection and maintenance. However, local linings and groutings may be used in any particularly leaky zones which may be encountered in order to control leakage into the reservoir.

Groundwater supplies in the Chicago metropolitan area are obtained from two aquifer systems. The "shallow aquifer" includes the glacial drift and Silurian-age bedrocks lying above the Makoqueta Shale as shown on Exhibit 10. The Makoqueta Shale is a practically impervious barrier between the shallow aquifer and the "deep aquifer" of which the Galena-Platteville Dolomite is the upper unit. The deep aquifer, which is used for most major groundwater supplies, must be protected from contamination from the underground reservoir. The shallow aquifer will be unaffected by conditions in the underground reservoir.

The piezometric level in the deep aquifer is presently higher than the underground reservoir location in the Galena-Platteville formation. However, the piezometric level is descending as much as 15 feet per year due to heavy pumping, particularly by suburban communities. It must be assumed, therefore, that the natural piezometric level will fall below the underground reservoir if not controlled. The descent of the piezometric level could be stopped either by restricting pumping or by artificial recharge of the aquifer.

It was concluded that artificial recharge of the deep aquifer would be the most likely method. Electric analog model studies were carried out to determine the amount of recharge water which would be required to maintain a pressure "mound" over the tunnel system as shown on Exhibit 11. The model study indicated an eventual requirement of 8 mgd (12 cfs), with initial recharging required in 1980. Seventeen or more recharge wells would be required to distribute the water to the aquifer in the project area. The calculated groundwater seepage inflow to the underground reservoir is 2 mgd (3 cfs).

Several sources of recharge water were considered, including filtered and chlorinated sewage treatment plant effluent, the groundwater supply of the shallow aquifer, and Chicago City water. Use of purified treatment plant effluent would be more costly in that both additional treatment facilities and a distribution system would be required, but use of this source would be in keeping with nationwide objectives of conservation of water. Transfer of groundwater from the shallow aquifer to the deep aquifer has the disadvantage that many more recharge wells would be required because of low well yields in the shallow aquifer. The cost of recharge water for this study has been based upon use of treatment plant effluent, purified to drinking water quality, assuming that the cost of tertiary treatment and chlorination would not be charged to the project.

Septic Conditions

Prevention of septic conditions in the upper and lower reservoirs has been a guiding criterion. The overflow waters which enter the underground reservoir will contain varying concentrations of sanitary sewage materials. Oxidation of these materials will take place while the overflow water is stored in the underground reservoir. If the overflows were to remain indefinitely in the underground reservoir, dissolved oxygen in the water would be consumed and anaerobic decay conditions would result. Reaction products of anaerobic decomposition include hydrogen sulfide gas and methane gas. Carbon dioxide, which dissolved in water forms carbonic acid, is produced by both aerobic and anaerobic decomposition.

It was judged that overflow waters could be held in the underground reservoir about four days before the onset of anaerobic conditions. However, under normal conditions, the overflow waters would be circulated between the underground reservoir and the surface reservoir almost daily in the course of power generation. This circulation would continually mix the overflow water with the water used for power generation and would give these mixed waters frequent opportunities for replenishment of dissolved oxygen from the atmosphere while in the surface reservoir and in the ventilated tunnels.

Organic solids and inorganic sediment would be deposited both in the underground reservoir and in the surface reservoir. Due to their lower specific gravity, some of the organic solids probably will be flushed out of the underground reservoir and deposit in the upper reservoir. The heavier, inorganic sands and gravels probably will remain largely in the underground reservoir. Data obtained from the Public Health Service investigations of the Roscoe Street sewer overflows indicate that in a typical year about 5500 cubic yards of sludge and grit would be deposited in the underground reservoir, representing 0.12 percent of the total reservoir volume. A parallel computation using influent quality data obtained at the Calumet Sewage Treatment Works gives an estimate of 10,500 cubic yards per year, or 0.23 percent of the total reservoir volume.

Organic solids deposited in the upper reservoir would be decomposed under aerobic conditions because of the frequent draining of the upper reservoir for power generation. Aerobic decomposition of these solids should present no problem, and the rate of storage encroachment by solids accumulation in the upper reservoir will be negligible.

The surface sewer relief facilities of the project will be designed to ingest most of the trash carried by the sewers without clogging the intake structures. Most of the trash materials will be pulverized in the vertical shafts and plunge pools and in passing through the pump-turbines. The costs associated with trash removal in the Deep Tunnel Plan will be considerably less than the cost of trash removal from the surface sewers.

Chapter IV

THE RECOMMENDED PLAN

Summary Description

The recommended plan, as noted in Chapter II, is the Deep Tunnel Plan 1C. This plan is a comprehensive underground reservoir system furnishing protection against 100-year floods to a service area of 21.0 square miles. The plan consists of 29.7 miles of new main sewers, 23.4 miles of 33-foot diameter tunnels about 800 feet below the land surface, a 300,000 kilowatt pumped-storage power plant, and a surface reservoir of 2360 acre-feet storage capacity. The system would have adequate treatment capacity to treat all storm-water inflows with the reserve possibility of passing polluted water through the Calumet Sewage Treatment Works. The total cost of this plan is estimated to be $85,200,000.

The layout of the recommended plan is shown on Exhibit 7. Details of the reversible pump-turbine station and the surface reservoir are shown on Exhibit 9. This plan would serve the area lying generally between Lake Calumet and Lake Michigan and between 87th Street and 138th Street. It would relieve existing lateral and main sewers at some 400 points. The reversible pump-turbine station and the surface reservoir would be located in a presently undeveloped area between 130th and 134th Streets and between the Calumet River and the Calumet Expressway.

Recommended Plan Components and Operation

The major components of the recommended plan were sized, located, and designed to a degree of accuracy adequate to allow the preparation of prefeasibility level cost estimates and to demonstrate the physical ability of construction and operation. Emphasis was placed on those elements which would most affect the overall practicability of the Deep Tunnel Plan. Such elements include the competence of the rock formations, the steps required to protect the aquifer, biochemical reactions in the underground and surface reservoirs, and the cost of underground excavation. Less emphasis was placed on items whose cost would be a small proportion of the total project cost or which are of a relatively conventional nature, such as the new main sewers, the power generating facilities, and the surface reservoir. Many design details will, of course, remain to be worked out in subsequent engineering studies and design.

The components of the recommended plan and their functions and operations are described in the following sections in a sequence which generally follows the direction of flow of storm water through the system.

Lateral Sewers

Existing lateral sewers and their associated house and street connections would be incorporated into the recommended plan. The discharge capacity of the existing laterals would, however, be substantially increased by relieving both the lower and the upper ends of each lateral. This would be accomplished either by a new main sewer or by a vertical drain direct to the underground system. Because the laterals have relatively flat slopes, about 40 percent of the total lateral flow would move against the normal sewer slope under surcharged conditions to the new relief points. The discharge capacity of the typical lateral would be increased from about six to about 15 cubic feet per second, a multiplication of 2.5 times. Some minor and infrequent basement flooding would remain but not enough in most cases to warrant the prohibitive cost of replacing the lateral sewers.

Profiles of selected representative streets showing the existing lateral and basement locations and elevations (estimated from surveyed first-floor elevations) are given in Appendix A. These profiles also show hydraulic grade lines of flows caused by storms of various recurrence intervals for the laterals under present conditions and as they would perform in the Deep Tunnel Plan. The hydraulic grade lines for laterals relieved by the underground plan result only from head losses in the lateral, while those for present conditions include backwater effects from surcharged main sewers as well as head losses in the laterals. The transport capacity of the relieved laterals-and-main-sewer system would be about 2.0 inches per hour as compared to the existing lateral-and-main-sewer system capacity of about 0.1 to 0.3 inches per hour, a multiplication of up to 20 times.

New Main Sewers

New main sewers would be required to carry relief flows from the upper ends of laterals and from selected points along present main sewers to vertical drains connecting to the underground reservoir system. These sewers would be similar to conventional sewers except that they would be laid deep enough to pass under existing utilities. Such sewers would have no gradient restriction since their outlets, being at vertical drains, would have no depth limitations. The new sewer sizes would range from 21-inch to 96-inch. These sewers would be generally laid in open cut, except where passing under such developments as expressways and railway yards.

The new main sewers would not receive dry-weather sewage flow, all of which would continue to be conveyed by the presently existing sewer system.

Cost comparisons were made of small diameter tunnel systems in the rock as substitutes for the required additional sewers. The tunnels and the extra vertical drains which would be required were found to be more expensive than the surface sewers provided in the plan.

Vertical Drains

Excessive flows in existing main sewers, together with overflows collected by the new main sewers, would be dropped to the deep underground reservoir system through some 68 vertical shafts averaging about 800 feet in length. These vertical drains, or spillways, would relieve all existing main sewers and those laterals not otherwise relieved to achieve the desired overall capacity of 2.0 inches per hour.

Intakes to the vertical drains would be designed to hold down the hydraulic grade line in the main sewers. The intakes would begin to operate when the main sewer was flowing about half full. The shape of the opening would be designed to minimize the possibility of trash blockage. Shaft diameters would range from 12 inches minimum to 56 inches with corresponding discharge capacities of about 30 cubic feet per second to about 1500 cubic feet per second. The high energy of the falling water would be dissipated in a plunge-pool sump located to the side of the tunnel into which the vertical drain discharged, to avoid damage to the tunnel invert.

The vertical drains also would serve as inlet points for ventilation induced by large exhaust fans at the pump-turbine station. The larger drains also could serve for personnel access during construction operations, thereby reducing the amount of underground travel time of the construction labor.

The smaller drains probably would be built by conventional water well drilling methods. The larger drains could be built most economically by ascending or descending boring machines. All drains probably would require steel lining throughout their length. Further description of the vertical drains and their hydraulic operation is given in Appendix A.

Underground Reservoir

The underground reservoir would consist of 23.4 miles of 33-foot diameter tunnel constructed by mole, furnishing 3,920,000 cubic yards of storage volume, plus 580,000 cubic yards of bulk excavation executed by drilling and blasting. Total underground reservoir volume would be 2820 acre feet. The tunnels and chambers comprising the underground reservoir would be unlined and generally unsupported. The Galena-Platteville dolomite formation, averaging 325 feet in thickness at a depth of 600 feet below the land surface, would be structurally adequate for such construction. The underground system would both store overflow water and convey it to the pump-turbine station.

Geologic investigations carried out for this prefeasibility study were based on data available in publications or in the files of State agencies and other institutions. The geologic section in the project area is shown on Exhibit 10. No test borings were made as a part of this study. However, the Galena-Platteville formations were inspected in quarry exposures near Rockford, Illinois, and core samples were inspected at the State Geological Survey offices in Urbana. It was found that the Galena-Platteville dolomite is exceptionally uniform, with a minimum of shale partings and chert inclusions. The strata are thick and highly dolomitized.

It was concluded that the Galena formation, averaging 200 feet thick, will be superior to the Platteville formation for the unsupported roof sections of the underground facilities, with free spans of up to 35 feet being possible. Roof sections should be arched at least one-third the span and rock bolting probably would be required in about 25 percent of the roof area. The formations would be adequate to support chambers up to 75 feet high, and no difficulty is contemplated in extending side walls of chambers across the Galena-Platteville interface. The geological setting and the characteristics of the individual formations are described further in Appendix B.

Alternative methods for excavation of the underground reservoir were investigated in considerable detail. The tunnel portion of the underground reservoir would be excavated with tunneling machines or "moles." The 580,000 cubic yards of bulk excavation would be carried out by drilling and blasting large galleries, using mining methods for greatest economy. Excavation methods and costs are described in Chapter V and in greater detail in Appendix F.

Future expansion of the first stage project underground reservoir is possible within the limits of the Galena-Platteville formation. The formation slopes downward at about 10 to 20 feet per mile in an east-south-east direction. All of the metropolitan area north and west of Lake Calumet could be gravity drained by tunnels or galleries on gradients parallel to the bedding surfaces.

Pumped-Storage Power

Water stored in the underground reservoir for drainage or power purposes would be evacuated by large reversible pump-turbines. Power would be generated by the same machines by discharging water from the upper reservoir to the lower reservoir. The capacity of the initial installation of machinery in the first stage of the project under the recommended plan would be 300,000 kilowatts while generating and 3750 cubic feet per second while pumping. The installation would consist of three pump-turbine, motor-generator sets in an underground machinery hall at about the same elevation as the lower reservoir. A cross-section sketch of the machinery hall is shown on Exhibit 9. The machinery would be designed to facilitate extension for installation of additional capacity as the drainage project was extended.

The underground machinery hall also would serve for access to the underground reservoir for inspection and maintenance. It would, however, be sealed from the underground reservoir during normal operation and have ventilation and drainage systems independent of those provided for the underground reservoir. Access from the surface would be provided by an elevator shaft separate from the shaft used for removal of excavated rock. This shaft also could be used for personnel access and equipment handling during construction operations. Vehicles and equipment for maintenance operations and sediment removal in the underground reservoir could be stored in the machinery hall.

Pumped-storage hydroelectric power would contribute to the power utility system both in supplying generating capacity for meeting peak power requirements and by maintaining an overnight load on large thermal generating units. The power utility would use the generating plant on a daily basis for contribution to daily peak loads, for emergency generation in case of a thermal unit breakdown, or as a reserve for sudden increases in load which could not be followed by slower-reacting thermal units.

One of the greatest advantages of a hydroelectric plan in a utility system is its ability to start quickly and follow rapid variations in power loads. Large thermal units require as much as 24 hours to go on the line from a cold start so that these machines must be kept on hot reserve to meet system peak loads. The efficient operation range of thermal generators is narrow, and it is desirable to operate them at as nearly constant load as possible. However, the power requirements vary widely during the daily cycle resulting in taxing of generating capacity during the day-time peak and surplus capability during the night-time valley. Pumped-storage plants, although consuming more energy than they produce, are an economical adjunct to a primarily thermal system because they consume low-cost, off-peak energy and produce valuable on-peak power.

The relationship of joint drainage and power use of the underground reservoir was studied for the 94-year period of Chicago rainfall records. It was concluded that 40 percent of the underground reservoir space could be made available for power generation without serious conflict between the uses. The underground reservoir volume is equivalent to 2.5 inches of runoff from the contributing drainage area, therefore, 1.5 inches of volume would be reserved for drainage use only.

Drainage volumes greater than 1.5 inches would have occurred only 13 times during the 94-year period.

The potential conflicts between power and drainage use of the underground reservoir were studied on an individual storm basis, using hourly rainfall values for the 1950-1965 period for calculating runoff volumes. These studies indicate a conflict on only three days in the 15 years. Power production could have been curtailed on two of these occasions without serious infringement on system power reserves and with no infringement on use of drainage space. In the remaining instance, minor flooding would have resulted from use of part of the space for power release unless storm forecasts had been utilized to curtail power generation.

After study of electrical system load data and load projections for the year 1975, an initial installed capacity of 300,000 kilowatts was selected as appropriate for the recommended plan. The total energy which could be generated in one daily cycle would be about 750,000 kilowatt hours, assuming a net head of 800 feet. The selection of installation capacities made in this study would be refined by more sophisticated methods of systems analysis in the feasibility studies.

The initial power installation could be readily increased if the underground reservoir is expanded to serve additional area. Desirable generating capacity would increase with time even if the reservoir size were not enlarged. As the system load peaks grow with time (the present rate of growth is about 7 percent per year), the amount of energy associated with the peak generation requirement decreases. Based on both of these factors, the ultimate generating capacity for this plant could be well over 2,000,000 kw. Further discussion of the recommended plan power facilities and of pumped-storage power principles is given in Appendix E.

Surface Reservoir

A reservoir at ground level is required to store water for generation of electricity. The first-stage power installation of 300,000 kilowatts proposed for the recommended plan would require a reservoir with 1120 acre feet of active storage capacity. Since extension of the first-stage project would accommodate additional generating facilities, the first-stage surface reservoir also would be designed for extension.

As the inclusion of power facilities in the project necessitates a surface reservoir and high capacity pumps, it becomes economical to provide for storage of some of the flood water in the surface reservoir. It was assumed in these prefeasibility studies that storage space for 1.1 inches of runoff would be provided in the surface reservoir. The total storage space provided would then be 2360 acre feet. An ungated concrete overflow spillway to the Calumet River south of O'Brien Locks would be provided to handle overflow above the combined storage level. This spillway would have had to operate once in the 94-year period of precipitation records.

The surface reservoir would be located in the presently vacant 500-to 600-acre tract of land south of 130th Street, between the Calumet River and the Calumet Expressway, as shown on Exhibit 9. The reservoir would be formed by rock-fill dikes which would be constructed using rock excavated from the underground reservoir. The impervious core would be constructed from glacial clays which occur in the reservoir area. The reservoir design used for initial-stage cost estimates occupies a land area of 145 acres with water stored to a depth of about 18 feet. The reservoir could be extended southward by the construction of a new dike. Alternatively, it could be built to its ultimate area in the initial stage and later extended by raising the dike. If the smaller-area reservoir were built in the initial stage, it would be advisable to acquire additional land for ultimate extension at the same time.

The reservoir site is underlaid by a 40-to 50-foot thick layer of stiff, impervious glacial clay which is well suited both for strength and resistance to seepage. Foundation conditions at the surface reservoir site are described further in Appendix B, and construction and costs are described in Appendix F.

Pollution Treatment

A basic objective of the Deep Tunnel Plan is to provide for treatment of storm-water overflows from combined sewers. As noted in Chapter II, it can be expected that the normal hydraulic operation of the system for drainage and pumped-storage power will furnish treatment more than adequate to bring the combined-sewer overflows to a high standard of quality. In addition, the storage capacity of the reservoirs in the system would make it possible, if necessary, to pass storm-water overflows through the Calumet Sewage Treatment Works at times of minimum normal demand on the Treatment Works.

It was calculated that storms of two inches or less rainfall are responsible for over 90 percent of the BOD (biochemical oxygen demand) which would enter the tunnel system (see Appendix C). Such storms would ordinarily have less than 0.7 inches of runoff, equivalent to about

270 million gallons. The Calumet Sewage Treatment Works is planned to have an ultimate capacity of 330 mgd (million gallons per day) of which 110 mgd will be reserved for storm flows. If this surplus capacity were made available for treatment of water from the Deep Tunnel Plan, runoff from a two-inch rainfall storm could be processed in less than three days.

Although provision for conveyance of waters from the surface reservoir to the Calumet Sewage Treatment Works is included in the cost of the recommended plan, it is expected that the treatment capacity of the Deep Tunnel Plan system itself will be more than adequate and that it actually may have spare capacity for other uses.

The components and the operations of the Deep Tunnel Plan closely parallel those of a conventional sewage treatment plant. Much of the trash entering the system will be pulverized in the descent through the vertical drains and in the violent turbulence of the plunge pools. Large pieces remaining would be screened out at the pump-turbine station. Primary sedimentation would take place in the underground reservoir. Aeration would take place in the vertical drains and in the surface reservoir and could be induced in the operation of the reversible pump-turbines. Final settling and clarification would take place in the surface reservoir.

The total amount of settleable solids which the recommended plan system will receive from storm-water overflows is estimated to be about 2100 tons per year, of which about 1700 tons would remain in the underground reservoir. The solids remaining in the underground reservoir would have a volume of about 10,000 cubic yards or 0.23 percent of the reservoir volume. Solids deposited in the underground reservoir could be readily removed during an annual maintenance period, using conventional loading and trucking equipment.

The rainwater origin of the bulk of the overflow water and air entrainment in the vertical drains would probably result in saturation of dissolved oxygen in water entering the underground reservoir. Under normal conditions, the storm-water overflows would enter the pumping-generating cycle of the pumped-storage power operation and circulate between the underground and surface reservoirs with the opportunity to replenish, at the surface, oxygen consumed in satisfaction of BOD.

Air is often introduced to water in both the pumping and generating operations of hydraulic machinery to reduce cavitation and to improve smoothness of operation. Large quantities of air, in addition to the inherent supply, could be introduced in this manner to enhance treatment.

Cycling of the storm-water overflows between the underground and surface reservoirs in the pumped-storage power operation will virtually eliminate the possibility of septic conditions in the underground reservoir. In addition, ventilation facilities would be provided to induce a constant flow of fresh air through the underground reservoir system and remove gaseous products of the treatment process.

Ventilation of the underground reservoir could be accomplished most readily by using the main construction access shaft as an exhaust point and the vertical drains as fresh-air introduction points. The large cross section of the main tunnels would result in little head loss so that the pressure differential across all the vertical drains would be about equal, and air inflow would be evenly distributed. For initial estimation of costs, it was assumed that exhaust capacity would have to be equal to double the maximum rate of water inflow, to be able to remove air displaced by inflowing water plus air entrained by the inflowing water. The requirements, location, and type of ventilation machinery will be further established in subsequent feasibility and design engineering studies.

Sanitary engineering aspects of the Deep Tunnel Plan are presented in greater detail in Appendix C.

Chapter V

PROJECT COSTS

The Deep Tunnel Plan is unique in concept but is made up of components that individually are conventional. The new main sewers are basically the same as conventional sewers except for greater freedom in gradient. The vertical drains would be bored shafts, cased by well-drilling methods. The underground reservoir compares in tunnel and chamber dimensions with existing tunnels and with mines, although the total volume would be unusually large. The underground, reversible pump-turbine station, the penstocks, and surface reservoir are all conventional hydroelectric project components.

Cost estimates were prepared for the project components and facilities to a degree of detail comensurate with this prefeasibility study. However, because of its great size and predominating influence on the total project cost, the cost of excavating the underground reservoir was calculated at a degree of detail normally reserved for feasibility-level studies. A comparison of component costs for the five plans considered is presented in Table V-1. Detailed cost estimates and discussions of the procedures used in estimating costs are presented in Appendix F.

Cost of Underground Excavation

Unit prices for underground reservoir excavation were studied for alternative construction methods; tunneling machine or "mole", and drilling and blasting. It was concluded that tunnel excavation would be most economically done by mole while excavation of large

TABLE V-1

COST SUMMARY

(Millions of dollars)

	(Service Area 21.0 sq. mi.)			(Comprehensive Project Service Area 29.6 sq. mi.)	
Item	Drainage Only (Plan 1A)	Plan 1A Plus Power (Plan 1B)	Comprehensive Project (Plan 1C)	"Mole" Construction (Plan 2A)	"Drill and Blast" (Plan 2B)
New main sewers	0.8	0.8	9.4	15.9	26.9
Vertical drains	3.9	3.9	2.7	5.4	2.2
Underground reservoir	36.8	26.9	26.7	37.8	43.6
Pump-turbine station	6.1	20.7	20.7	26.1	26.1
Ventilation	1.0	1.0	1.0	1.4	1.4
Surface reservoir	----	4.7	4.7	5.7	5.7
Pipeline to treatment plant	0.8	0.8	0.8	0.8	0.8
Aquifer recharge	2.6	2.6	2.6	2.6	2.6
Subtotal	52.0	61.4	68.6	95.7	109.3
Contingencies - 15%	7.8	9.1	10.3	14.3	16.4
Total Direct Cost	59.8	70.5	78.9	110.0	125.7
Engineering and Overhead - 8%	4.8	5.5	6.3	8.8	10.1
TOTAL	64.6	76.0	85.2	118.8	135.8

Tunneling Machine - "Mole"

The mole, although a relatively new machine, has rapidly become the conventional method for tunneling in homogeneous rock. The mole is essentially a large rotary rock drill. The entire front face of the machine is made up of a rotating disc studded with rock bits. As the disc rotates, the entire machine is pushed forward by jacks bearing on the tunnel walls. Rock cuttings, ranging in size up to five inches, are passed through the machine by conveyor for transfer to hauling equipment.

Moles operating in rock similar to the Galena-Platteville dolomite have achieved advance rates of over 100 feet per day. An advance rate of 56 feet per day was used in preparing this cost estimate. Use of a 13-foot diameter mole of a relatively primitive design was attempted unsuccessfully for construction of the 79th Street water tunnel in 1962. However, this same machine was subsequently used successfully in limestone rock in Toronto, after ripper teeth, adapted to softer rock, were replaced by hard rock rolling bits.

Underground excavation costs for the 23.4 miles of 33-foot diameter tunnels in the recommended plan were calculated, assuming the use of two mole machines. The total cost of the moles and the total cost of all other construction equipment was assumed to be amortized over the course of the project, with no credit given to salvage value. Underground transport of spoil was assumed to be by mine railway to high-speed mine hoists in two 30-foot diameter access shafts.

The excavated rock probably would have substantial value as a construction material since it is of high quality and is produced in an effectively crushed condition. However, for purposes of conservatism, it was assumed that rock not used in construction of the upper reservoir would be taken from the shaft head for other uses at no cost. The unit cost of rock excavation by mole in the recommended plan, totaling 3,920,000 cubic yards, was calculated to be $6.00 per cubic yard.

In addition to its excavation economy, the mole has several other advantages. The rate of advance is much greater than the rates possible by drilling and blasting. Working conditions are safer, no fumes are produced, blasting shock and noise are eliminated, and the excavated rock is produced in a convenient size for handling. The tunnel produced by the mole, being circular and smooth, has idea hydraulic and structural characteristics. The resulting tunnels have no overbreak; and, since no shattering of the rock occurs, the need for structural supports is minimized.

Drilling and Blasting

Although it was concluded that excavation by mole would have many advantages, the more traditional method of drilling and blasting also was investigated in order to provide comparative data on costs. Drilling and blasting was found to be more expensive than mole excavation for tunnels. Unit prices of the drill-and-blast method were estimated to range from $40 per cubic yard for ten-foot diameter tunnels to $14.50 per cubic yard for 35-foot diameter. However, excavation by drilling and blasting was found to be more economical than mole for excavation of bulk storage requirements over and above the volume of the tunnels.

The greatest economy in excavation by drilling and blasting can be achieved by excavating large volumes of rock in close proximity to the removal shaft. Accordingly, the 580,000 cubic yards of volume required in the recommended plan that is not provided by tunnels was assumed to be excavated in the form of two large parallel galleries immediately north of the pump-turbine station. These galleries were assumed to be 32 feet wide and 75 feet high, spaced about 150 feet on centers. Two galleries would be excavated so that drilling and loading operations could proceed continuously on alternate faces. The galleries for the recommended plan would extend northward about 3000 feet from the pump-turbine station.

Detailed studies of construction procedures, schedules, and equipment were made and are described in Appendix F. The unit price for bulk excavation by drilling and blasting was calculated to be $5.50 per cubic yard.

The total cost estimated for the 4,500,000 cubic yards of underground reservoir excavation in the recommended plan was estimated to be $26,700,000. However, it is probable that the total cost could be reduced by determining the optimum relationship of volumes excavated by the different methods. This would be done by methods of systems analysis in feasibility and design studies.

The estimates of excavation costs were reviewed in detail by Paul Weir Company, mining engineering consultants. The construction procedures and costs also were discussed with a prominent tunnel contractor in the Chicago area and with a major manufacturer of mole tunneling machines.

Cost of Other Plan Components

Cost estimates for components of the plan, other than the underground reservoir, were generally based on cost experience from similar projects.

New Main Sewers

Construction of the new main sewers would be by conventional methods although they would be laid somewhat deeper, on the average, to pass under existing utilities. Gradients would also be steeper, resulting in higher velocities and smaller pipe sizes.

Unit prices were based on similar work, including Sanitary District experience. Sewer sizes and unit prices range from 21-inch diameter at $25 per lineal foot to 96-inch diameter at $130 per foot. Manhole prices range from $550 for standard 42-inch or smaller sewer to $14,000 for regulator-controlled relief spillways. The total cost for new main sewers and associated manholes, weirs, and regulators for the recommended plan was estimated to be $9,400,000.

Vertical Drains

Vertical drains in the recommended plan would range in size from 12-inch diameter to 56-inch diameter, and would average about 800 feet in length. The drains were assumed to have steel casings for their full length. Costs were estimated on the basis of conventional rotary or percussion well drilling and casing methods. Costs include head chambers and plunge pool stilling basins and range from $20,000 for 12-inch drains to $105,000 for 56-inch drains. The total cost for vertical drains in the recommended plan was estimated to be $2,700,000.

Pumping-Generating Station

The cost of the pumping-generating station, shown on Exhibit 9, would include the excavation of main and auxiliary chambers; concrete arch roofs and concrete substructures; the waterway facilities such as penstocks, draft tubes, gates, and valves; the reversible pump-turbine -- motor-generator units; and the associated mechanical and electrical equipment. Costs for these items, listed in detail in Appendix F, were estimated principally on the basis of construction experience with similar projects.

The total cost of the 300,000 kilowatt pumping-generating station of the recommended plan was estimated to cost $20,700,000. The price per kilowatt installed would be about $70 which falls within the normal range for an underground powerhouse of this type.

Surface Reservoir

The surface reservoir dikes would be made from rock excavated from the underground reservoir, with impervious cores made of local glacial clay. The rock fill would require only placing cost, as excavation cost includes hoisting to the surface. Land in the reservoir site area is zoned industrial and has been assumed to cost $20,000 per acre. The total cost for the 2360 acre-foot surface reservoir in the recommended plan was estimated to be $4,700,000.

Contingency and Overhead

The costs estimated for the plan components are total direct costs including a 32.5 percent allowance for contractors' profit and overhead. A contingency allowance of 15 percent was added to the total of direct costs. This contingency intended to account for variations in prices, unforeseen construction and foundation problems, and omissions. The contingency is less than the 20 percent usually used in prefeasibility studies because of the more detailed study made of excavation costs, the largest single item, and because of the conventional nature of items such as the new main sewers, power station, and surface reservoir. In addition no contingency is necessary for possible increase in volume of underground construction (a frequent cause of additional costs) since the end objective is to provide volume and, if overbreak occurs at one location, appropriate reductions in volumes can be made at other locations.

An allowance of eight percent was added to the total of direct cost and contingency to account for engineering, overhead, and administration costs.

Chapter VI

JUSTIFICATION OF THE DEEP TUNNEL PLAN

General Method of Evaluation

The principle benefits of the Deep Tunnel Plan fall into three categories: pollution abatement, urban drainage, and power production. Of these, only the power benefits are susceptible to detailed monetary evaluation.

An economic evaluation of the pollution abatement and urban drainage benefits is not essential for the purpose of this report. Decisions to finance projects for such purposes have traditionally been based on the need for achieving certain standards of pollution control and flood protection, rather than on a benefit-cost analysis. Since the purpose of this report is to allow a decision on the expenditure of funds for detailed engineering studies. rather than on financing, it is evident that benefit-cost analyses are not needed at this stage.

Under this approach, the most important consideration in evaluation of the Deep Tunnel Plan is its cost as related to alternative methods of providing the same benefits. To make this comparison, the alternative capital cost of the only revenue-producing feature (power) has first been deducted from the total cost of the Deep Tunnel Plan. The residual cost is then compared to the cost of a drainage-only plan (the Conventional Plan). This is followed by a qualitative comparison of residual benefits.

Comparison of Alternative Costs for Flood Control

The primary value of the hydroelectric pumped-storage facilities is in replacing alternative sources of generation during peak power load periods. There are supplementary benefits in providing a pumping power load during periods of low system load and in providing a ready reserve for quick starting in emergencies. However, these have a lower economic value than the replacement of alternative peaking capacity.

A range in cost of alternative peaking capacity is used for the preliminary evaluation objective of this report. It is recognized that detailed economic evaluations will require consideration of a number of factors, such as relative operation and maintenance costs, costs of pumping energy, and the supplementary benefits of pumped-storage operation mentioned above. This detailed evaluation is left for the feasibility stage of study.

It is doubtful if peaking capacity can be obtained for as low as $70 per kilowatt. At the other extreme, pumped-storage plants have been built by private utilities at costs of over $110 per kilowatt. On the basis of this range, the investment cost of an alternative to the 300,000 kilwatts of capacity associated with the proposed Deep Tunnel Plan would range from $21,000,000 to $33,000,000. Subtracting these values from the total project cost of $85,000,000 leaves a residual cost of between $52,000,000 and $64,000,000.

The latest available estimate of alternative cost of providing drainage only by the Conventional Plan is about $62,000,000. On this basis, the Deep Tunnel Plan appears highly promising, particularly when the incremental benefits offered by that plan, over and above those available from the Conventional Plan, are considered. These incremental benefits may be summarized as follows:

(a) The Deep Tunnel Plan will allow storage and secondary treatment of overflow storm water, which is not possible in the Conventional Plan unless surface storage and conveyance to the treatment plant are added.

(b) It will provide the difference in flood protection and drainage offered by a design objective for a 100-year flood as compared to the objective of providing for a five-year flood in the Conventional Plan, and it will provide relief to more existing sewers than those to be relieved by the Conventional Plan as now conceived.

(c) It will provide a potential for tertiary treatment of sewage plant effluent and for spare capacity for secondary treatment.

(d) It will add operating flexibility and efficiency to the sewage treatment and storm flow system because of the vastly enlarged storage available underground.

The incremental benefits from the Deep Tunnel Plan are discussed in the following paragraphs. Greater detail is given in Appendix G.

Pollution Abatement Benefits

General Description

Benefits from control of pollution in the waterway system may arise from the following:

(a) Improvement in environment, including recreational use, pleasure boating, and enhancement of shore properties.

(b) Improvement in quality of water supplies for municipalities and industries utilizing the waterways.

(c) Enhancement of fish and wildlife propagation.

(d) Reduction in public health hazards.

The Deep Tunnel Plan, even if expanded to the entire Chicagoland area, will not by itself relieve pollution in the entire waterway system. If sources other than storm water overflows which contribute pollution are not similarly treated, the above benefits will not accrue to any appreciable portion of the waterway system. It is significant to note, however, that improvements to eliminate waste sources other than from combined sewers are scheduled or already under way. The Deep Tunnel Plan would be a first step toward elimination of pollution from combined sewer overflow.

The above reasons, as well as the inherent general difficulty of evaluating pollution control, preclude the assignment at this time of monetary values to direct benefits from control of pollution. The costs of alternative methods which might be available to control pollution could be considered as a measure of the pollution abatement benefit of the Deep Tunnel Plan if the alternatives are assumed to be economically justified.

Alternative Costs of Pollution Abatement

Storage and Treatment of Combined Overflows. This method has potential for complete solution of the overflow problem. This might be accomplished in the Calumet area under the Conventional Plan by construction of a surface reservoir similar to that in the Deep Tunnel Plan. This would then provide the same degree of abatement as the Deep Tunnel Plan; but, as a minimum, the cost of the reservoir ($4,700,000) would have to be added to the alternative project cost. In addition , it is considered very probable that the Deep Tunnel Plan will provide an additional benefit through secondary treatment by circulation in the reservoir system, without passage through the Calumet Sewage Treatment Plant. Thus, the alternative cost of such treatment could be considered as an added cost to the Conventional Plan to provide the same treatment as the Deep Tunnel Plan. On the basis of present minimum costs of BOD removal, the anticipated BOD removal by the Deep Tunnel Plan would have an additional incremental capitalized benefit of at least $5,000,000.

Separation of Storm and Sanitary Sewers. Separation would reduce but not eliminate waterway pollution, because storm runoff from urban areas is itself significantly polluted. It has been estimated by the U. S. Public Health Service that separation in Chicago would cost $17,000 per acre, which would amount to about $225,000,000 for the 21.0 square miles in the Calumet area. This great cost would be accompanied by great inconvenience and disruption of traffic because of excavations required in most of the streets.

Another scheme of separation, conceived by Professor Gordon Fair, is presently being studied by an American Society of Civil Engineers task group. That plan would have domestic sewage passed through a grinder in each building and pumped through a small plastic pipe, inserted in the existing combined sewer, to an interceptor for sanitary sewage. The existing combined sewers would carry storm water to the overflow points. Investigations of this method are now being initiated and costs have not yet been developed.

Tertiary and Spare Treatment Capacity. The possibility exists in the Deep Tunnel Plan of dropping effluent from the Calumet Treatment into the underground reservoir.. The effluent could then be recirculated in the system, providing the equivalent of tertiary treatment. Also the plan offers the possibility of increased BOD removal capacity by injection of air at the turbines during power generation and pumping, thereby providing the equivalent of spare treatment capacity. Potential benefits of these types remain to be investigated in detail, and can at this time neither be stated as certainties nor be compared to alternatives of known estimated costs. Appendix C contains a brief discussion of these possibilities.

Flood Control Benefits

Flood control and drainage benefits include the value of flood damages prevented and the enhancement of property value after removal of flood risks. Intangible benefits also result from removal of risks such as impairment by floods of fire and police protection, disrupted schooling, health hazards, and delays and inconveniences.

Basement Flooding

Basement flooding is the principal hazard in the Calumet area. Damage to property and utilities in basements and to the structure itself is common. In addition, the usefulness of basements for recreation, living space, and other beneficial uses is often restricted by the ever-present threat of flooding.

An estimate of basement flood damages was made, utilizing anticipated flood frequencies and Corps of Engineers' procedures for damage evaluation. The capitalized value of average annual losses was estimated to be about $11,000,000. An attempt to check this estimate was made by questionnaires sent to a significant sample of residents of the area. The results obtained from the questionnaire tended to support the underlying assumptions of the estimate by the Corps of Engineers' method. While complete agreement was not obtained to fully substantiate the estimate, the value of $11,000,000 can be considered as being of the proper order of magnitude.

The design objective of the Deep Tunnel Plan is to protect against a 100-year flood, as compared to the design objective of a five-year flood for the Conventional Plan. While neither Plan accomplishes the complete objectives under all conditions, the relative objectives are valid as the basis for comparison of the two plans. From this it can be seen that substantially greater flood protection benefits will accrue to the Deep Tunnel Plan.

Flooding of Industrial and Other Areas

Data are not available on the extent of flooding on the 4400 acres of industrial area in the area encompassed by the recommended initial development of the Deep Tunnel Plan. However, some flooding of these areas undoubtedly exists. Such floods will be reduced by a greater amount by the Deep Tunnel Plan than by the Conventional Plan. The same applies to street and underpass flooding.

Land Enhancement

There will be appreciable benefits accruing to both the Conventional Plan and the Deep Tunnel Plan through enhancement of vacant land values for which drainage would be provided under either plan.

The elements of land enhancement include, (a) the increase in value of some 3,200 vacant lots in existing residential developed area, (b) the increase in value of 2,700 acres of presently undeveloped areas that are zoned for residential and commercial use, and (c) reduction in local sewerage costs in presently unsewered areas.

The magnitude of land enhancement values has not been computed; however, they certainly would be quite significant. Land enhancement would have to provide a large portion of the benefits occurring to the Conventional Plan for that plan to be justified. The incremental benefits occurring for the Deep Tunnel Plan, over and above those obtained by the Conventional, also would be significant, because of the greater degree of protection afforded.

Flood Reduction on Waterway System

The proposed Deep Tunnel Plan would reduce flood outflows from the 21.0 square miles of drainage area from a maximum of 5000 cfs to negligible proportions. There would be no reduction offered by the Conventional Plan - in fact, flood outflows to the Illinois Waterway system probably would be increased.

The value of reductions of outflow to the waterway system has not been estimated. However, the significance of flood reduction may be exemplified by the fact that the Corps of Engineers and the State have recently announced a plan for the entire system which would cost about $500,000,000. That plan has not been approved, but the issuance of such proposals indicates a seriously considered value of flood reductions on the waterway.

Conclusions

The results of the prefeasibility studies demonstrate that the Deep Tunnel Plan is highly promising. The preliminary comparisons indicate it is more favorable than the Conventional Plan. It deserves investigation in the detail to which studies of the Conventional Plan have been authorized. These conclusions merit expenditures of funds for studies and investigations adequate to determine full engineering and economic feasibility of the Deep Tunnel Plan.

Chapter VII

RECOMMENDED INVESTIGATION PROGRAM

In view of the conclusions of these prefeasibility studies - that the underground reservoir storage concept has great potential economic merit and that no limitations to the concept were found to exist - it is recommended that feasibility-level studies of the proposed project be initiated.

The recommended feasibility studies consist of engineering studies and analyses, together with field investigations carried out in sufficient detail to establish full engineering, economic, and financial feasibility of the Deep Tunnel Plan. These studies would be carried out in sufficient detail to support requests for funds for construction of the initial phase of the Plan. The recommended field investigations would consist primarily of subsurface exploration of the geological structure in the project area and of groundwater conditions in and around the project area.

Recommended engineering studies and activities for the feasibility investigations are given in the following sections:

Hydrologic Studies.

(a) Hourly studies of critical rainfall-runoff events.

(b) Neighborhood drainage system characteristics.

(c) Lateral and main sewer flow capacities and characteristics.

(d) Studies and model test of new main sewers, spillways, and vertical drains.

(e) Analysis of the hydraulic performance of the underground reservoir for drainage and power purposes.

(f) Determination of the effects of the Plan on flood stages and flows in the waterway system.

Sanitary Engineering Studies.

(a) Study of potential health hazards associated with flooding of basements and streets and pollution of the waterways.

(b) Determination of the quality of combined sewer overflows as related to quantity and duration.

(c) Determination of chemical and biological reactions in the sewers, vertical drains, underground reservoir, pumping-generating facilities, and the upper reservoir.

(d) Calculation of potential quantity of trash and sediment inflow and methods for control and removal.

(e) Determination of requirements, methods, and facilities for treatment of overflow water, particularly the possible reduction in BOD of storm water overflows by inherent and induced aeration in the upper reservoir and in the pump-turbine machinery.

(f) Determination of the extent and nature of pollution abatement in the receiving waterways attributable to the Plan.

Geological Studies.

(a) Further evaluation of existing data.

(b) Preparation, administration, supervision, and interpretation of a drilling and testing program.

(c) Preparation of engineering criteria for the location and design of all physical components of the Plan.

Groundwater Studies.

(a) Further evaluation of existing well.data.

(b) Preparation, administration, supervision, and interpretation of an aquifer test program which would include a recharge test well, observation wells, and pump-test wells.

(c) Preparation of a more detailed electric analog model of the aquifer system.

(d) Determination of water table migration, recharge requirements, and groundwater inflow quantity into the underground facilities.

Hydroelectric Power Studies.

(a) Selection of optimum installed capacity.

(b) Analyse relationship of use of the underground reservoir for drainage and power based on hourly study of all critical runoff events.

(c) Layout and preliminary design of the power facilities and machinery.

(d) Study of the value and cost of power generation.

Project Layouts.

(a) Combine criteria and requirements of above studies in layouts of the physical components of the Project toward the objectives of minimum cost and maximum utility.

(b) The methods of systems analysis would be used to obtain optimum layouts and sizes of the Plan and its components taking into account the inter-relationship of components in both function and cost.

Cost Estimates.

(a) Further substantiation of unit costs developed in the prefeasibility studies.

(b) Determination of total Plan cost, including the costs of new sewers, sewer modifications, vertical drains, underground excavation, pumping-generating facilities, surface reservoir, groundwater recharge, and water treatment.

Economic and Financial Studies.

(a) Determination of tangible benefits attributable to reduction of basement flooding and other local flooding, reduction of flooding in the waterways, and hydroelectric power production.

(b) Determination, description, and evaluation of intangible benefits.

(c) Determination of secondary benefits and costs to the community attributable to the Plan.

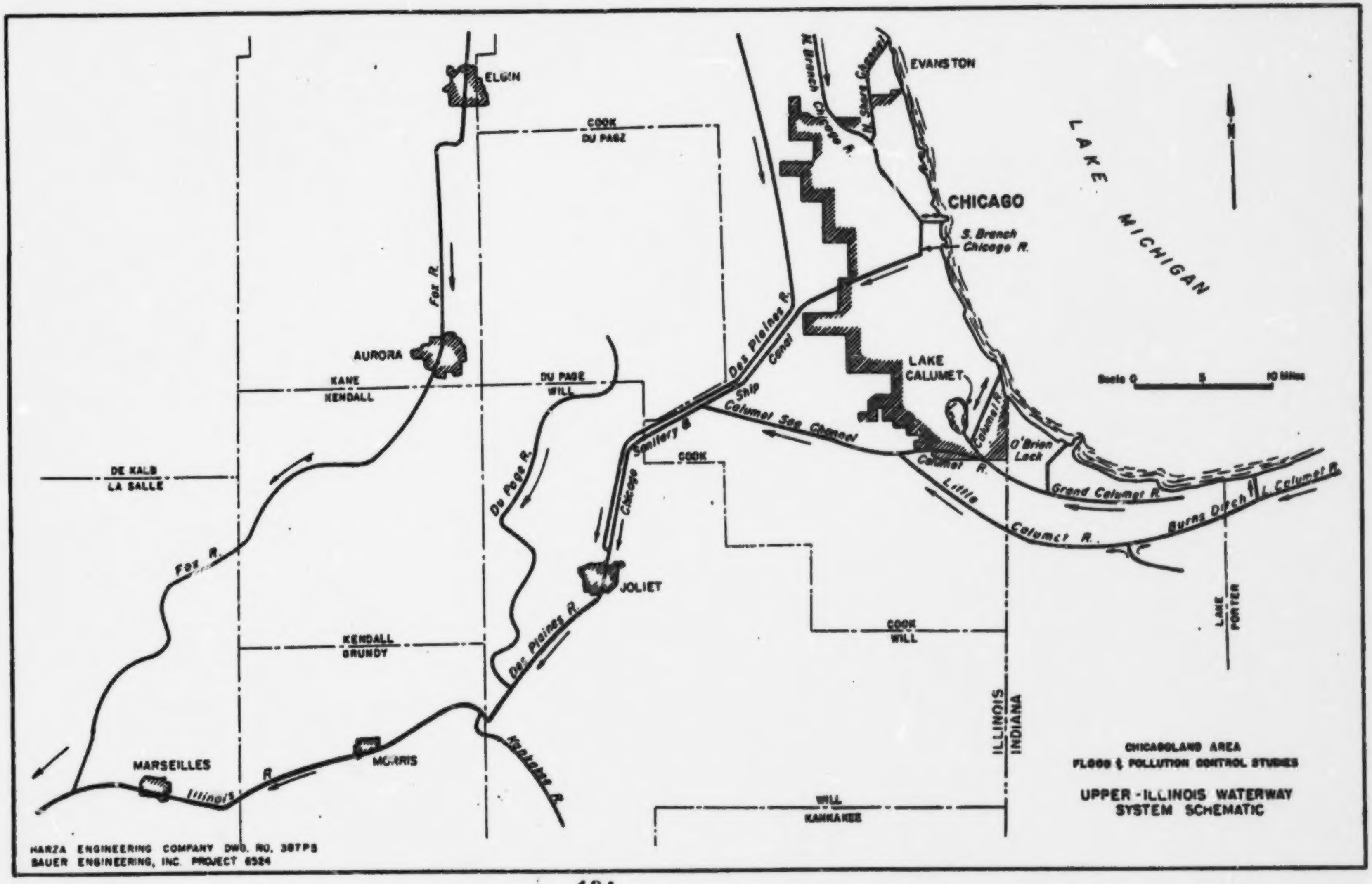
(d) Preparation of schedules and rates for construction and operation.

(e) Negotiations with financing institutions.

EXHIBITS

1. Upper-Illinois Waterway System Schematic
2. Existing River Pollution
3. General Concept
4. Drainage System Elements
5. Land Use and Drainage Boundaries
6. Conventional Plan
7. Recommended Plan
8. Alternative Plan
9. Surface Reservoir and Pump-Turbine Station
10. Geologic Map and Sections
11. Aquifer Protection

EXHIBIT I

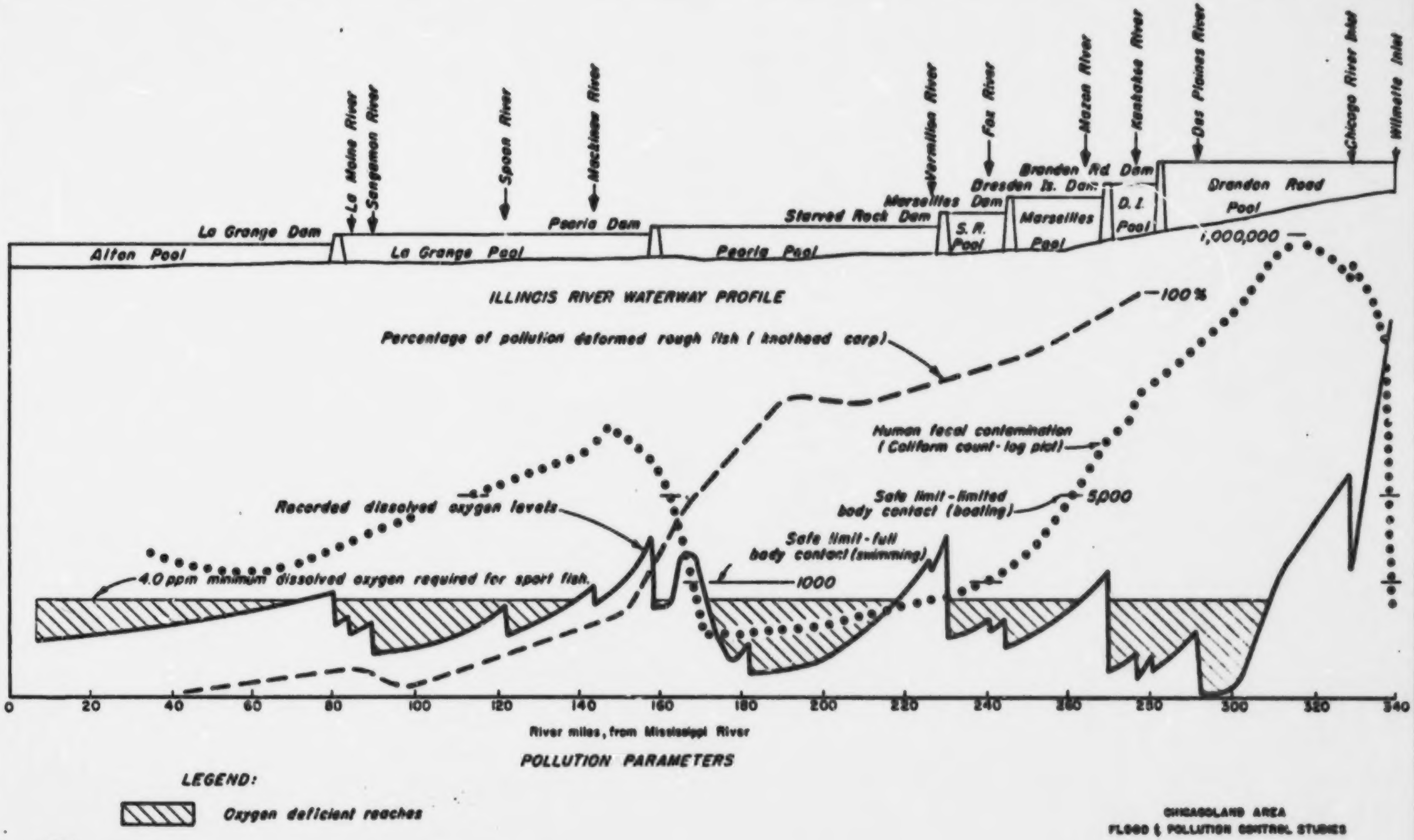

ELGIN
COOK
DU PAGE
EVANSTON
N. Branch Chicago R.
N. Shore Channel
LAKE MICHIGAN
N
CHICAGO
S. Branch Chicago R.
Fox R.
AURORA
KANE
KENDALL
DU PAGE
WILL
Des Plaines R.
Ship Canal
Chicago Sanitary & Ship Canal
LAKE CALUMET
Calumet R.
Calumet Sag Channel
O'Brien Lock
Scale 0 5 10 Miles
DE KALB
LA SALLE
COOK
Calumet R.
Grand Calumet R.
Little Calumet R.
Burns Ditch
L. Calumet R.
Du Page R.
Fox R.
JOLIET
Des Plaines R.
LAKE
PORTER
COOK
WILL
KENDALL
GRUNDY
ILLINOIS
INDIANA
Kankakee R.
MARSEILLES
Illinois R.
MORRIS
WILL
KANKAKEE

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

UPPER-ILLINOIS WATERWAY
SYSTEM SCHEMATIC

HARZA ENGINEERING COMPANY DWG. NO. 387PS
BAUER ENGINEERING, INC. PROJECT 6324

EXHIBIT 2



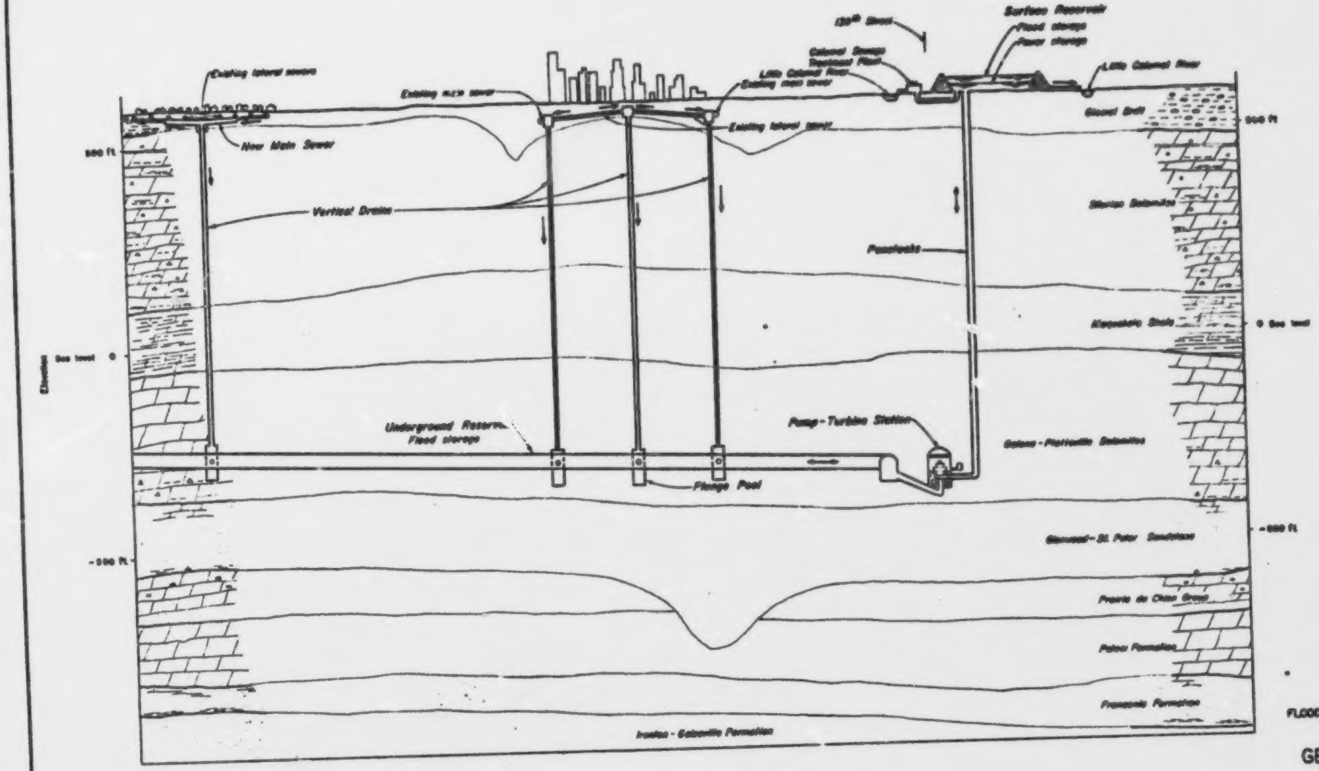
NOTE:
Data from Illinois Natural History Survey (Starrett, et al, 1966)
and U.S. Public Health Service.

HARZA ENGINEERING COMPANY DWG. NO. 387P-8
BAUER ENGINEERING, INC. PROJECT 6524

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

EXISTING RIVER POLLUTION

EXHIBIT 3

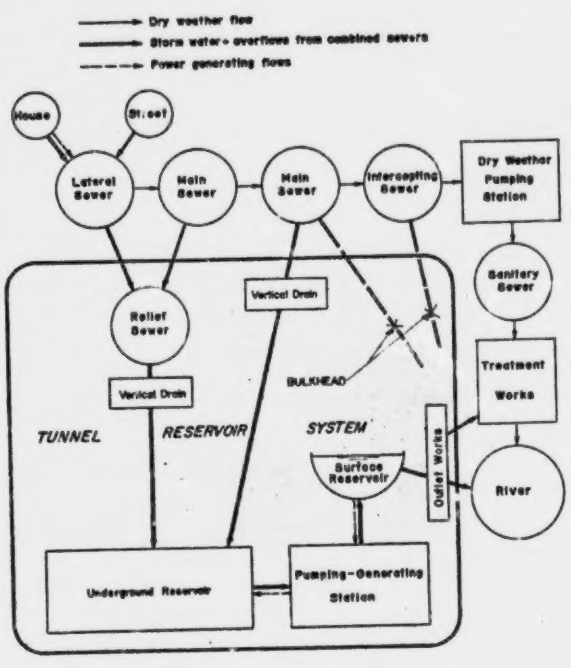

Existing lateral sewers
New Main Sewer
Vertical Drains
Existing main sewer
Existing lateral sewer
Little Calumet River
Existing main sewer
Calumet Sewage Treatment Plant
130th Street
Surface Reservoir
Flood storage
Power storage
Little Calumet River
Penstocks
Underground Reservoir
Flood storage
Plunge Pool
Pump-Turbine Station
Glacial Drift
Silurian Dolomites
Maquoketa Shale
Galena-Platteville Dolomites
Glenwood-St. Peter Sandstone
Prairie du Chien Group
Potosi Formation
Franconia Formation
Ironton-Galesville Formation
500 ft.
Elevation
Sea level 0
-500 ft.
500 ft
0 Sea level
-500 ft

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

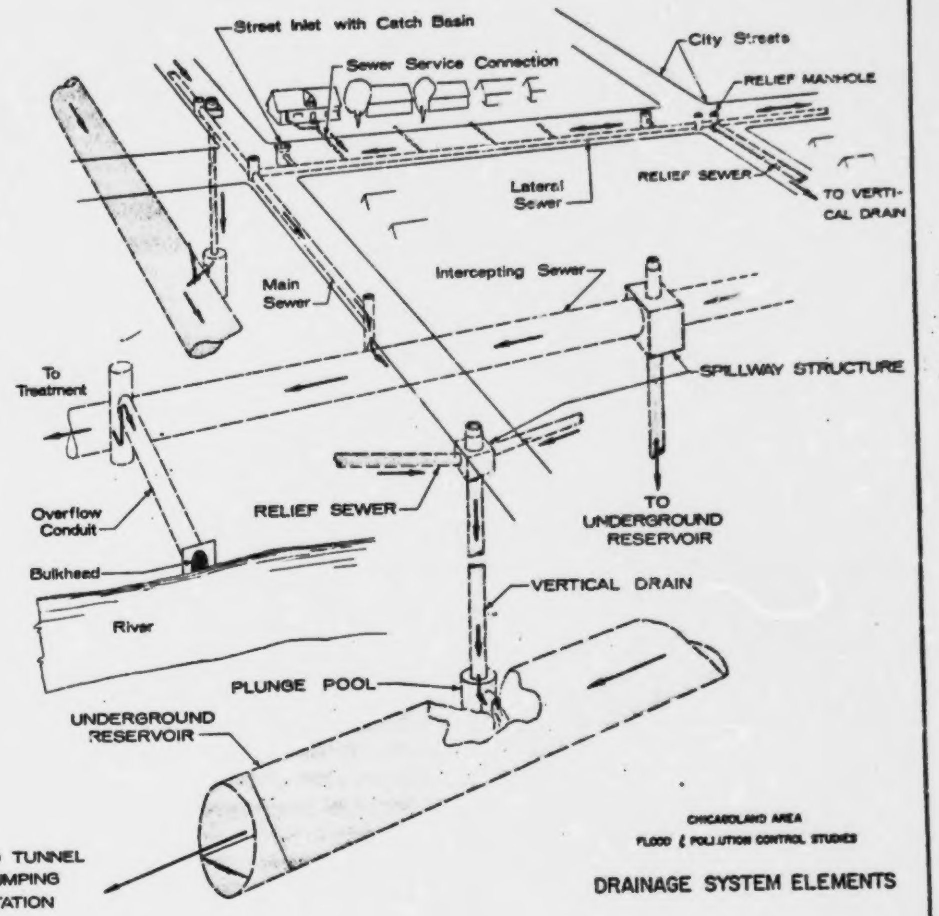
GENERAL CONCEPT

HARZA ENGINEERING COMPANY DWG.NO.387 P-1
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT 4


Dry weather flow
Storm water + overflows from combined sewers
Power generating flows
House
Street
Lateral Sewer
Main Sewer
Main Sewer
Intercepting Sewer
Dry Weather Pumping Station
Sanitary Sewer
Relief Sewer
Vertical Drain
Vertical Drain
BULKHEAD
Treatment Works
TUNNEL
RESERVOIR
SYSTEM
Surface Reservoir
Outlet Works
River
Underground Reservoir
Pumping-Generating Station

FLOW SHEET-PROPOSED SYSTEM

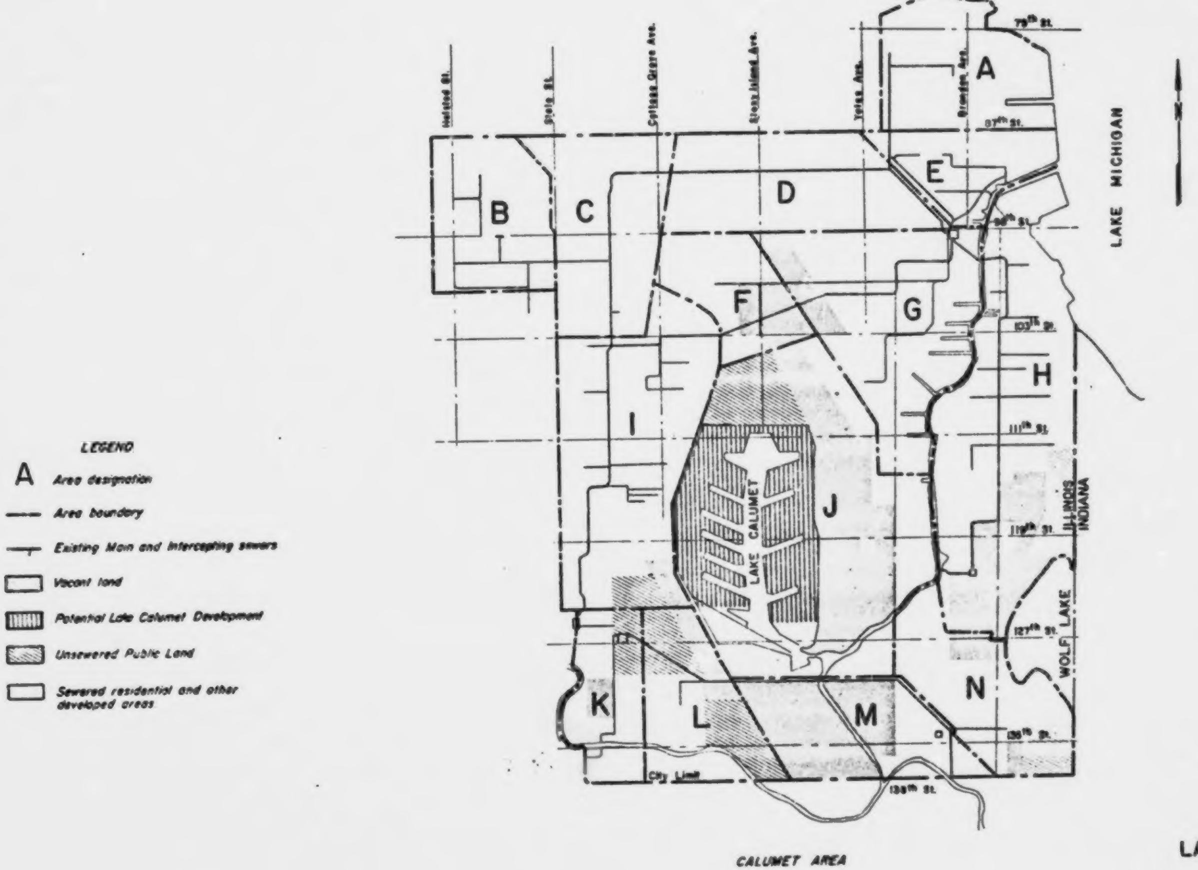

Street Inlet with Catch Basin
Sewer Service Connection
City Streets
RELIEF MANHOLE
RELIEF SEWER
TO VERTICAL DRAIN
Lateral Sewer
Main Sewer
Intercepting Sewer
SPILLWAY STRUCTURE
To Treatment
Overflow Conduit
RELIEF SEWER
TO UNDERGROUND RESERVOIR
Bulkhead
VERTICAL DRAIN
River
PLUNGE POOL
UNDERGROUND RESERVOIR
TO TUNNEL PUMPING STATION

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

DRAINAGE SYSTEM ELEMENTS

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT #324

EXHIBIT 5


LEGEND
A Area designation
Area boundary
Existing Main and Intercepting sewers
Vacant land
Potential Lake Calumet Development
Unsewered Public Land
Sewered residential and other developed areas
Halsted St.
State St.
Cottage Grove Ave.
Stony Island Ave.
Yates Ave.
Brandon Ave.
79th St.
87th St.
95th St.
103rd St.
111th St.
119th St.
127th St.
130th St.
138th St.
A
B
C
D
E
F
G
H
I
J
K
L
M
N
LAKE CALUMET
LAKE MICHIGAN
ILLINOIS
INDIANA
WOLF LAKE
City Limit
CALUMET AREA
N
Scale 0 1 Mile

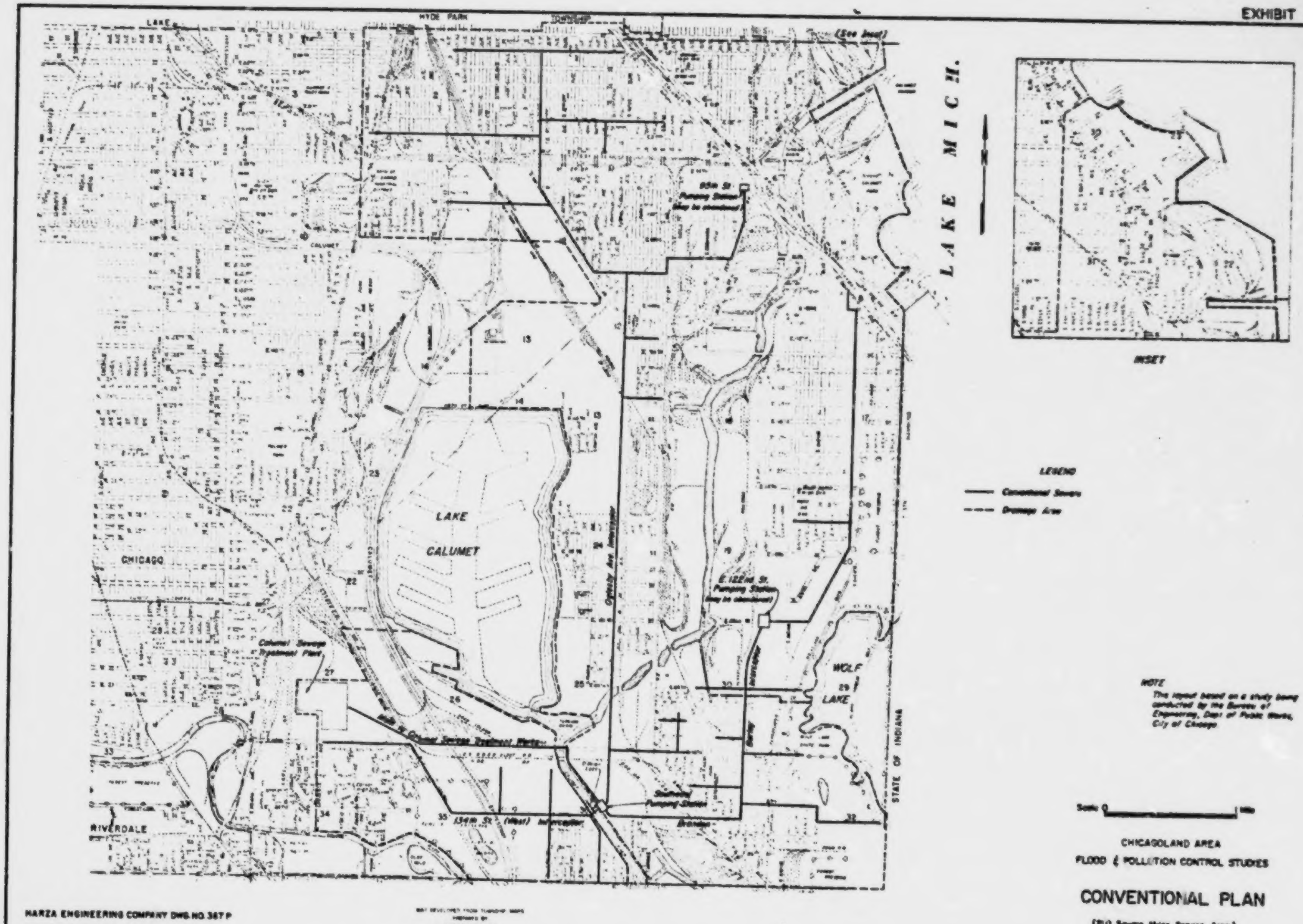
EXISTING SEWER OUTLET CAPACITIES			
AREA	ACRES	OUTLET, cfs	cfs/Ac
A	1200	200	0.17
B	1150	120	0.10
C	1420	360	0.25
D	1470	220	0.15
E	730	80	0.11
F	800	50	0.06
G	2080	200	0.10
H	2800	500	0.18
I	2400	320	0.13
J	4200	—	—
K	730	60	0.08
L	1080	—	—
M	1200	—	—
N	1150	120	0.10

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

LAND USE AND DRAINAGE BOUNDRIES

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT NO. 6324

EXHIBIT 6


LAKE
HYDE PARK
TOWNSHIP
(See Inset)
LAKE MICH.
INSET
95th St. Pumping Station (May be abandoned)
LAKE CALUMET
CHICAGO
CALUMET
Calumet Sewage Treatment Plant
Calumet Sewage Treatment Works
Ewing Ave. Interceptor
E. 122nd St. Pumping Station (May be abandoned)
Stateline Pumping Station
134th St. (West) Interceptor
WOLF LAKE
STATE OF INDIANA
RIVERDALE
LEGEND
Conventional Sewers
Drainage Area
13
14
15
19
20
22
23
24
25
26
27
29
30
32
33
34
35
Scale 0 1 Mile

NOTE
The layout based on a study being conducted by the Bureau of Engineering, Dept of Public Works, City of Chicago

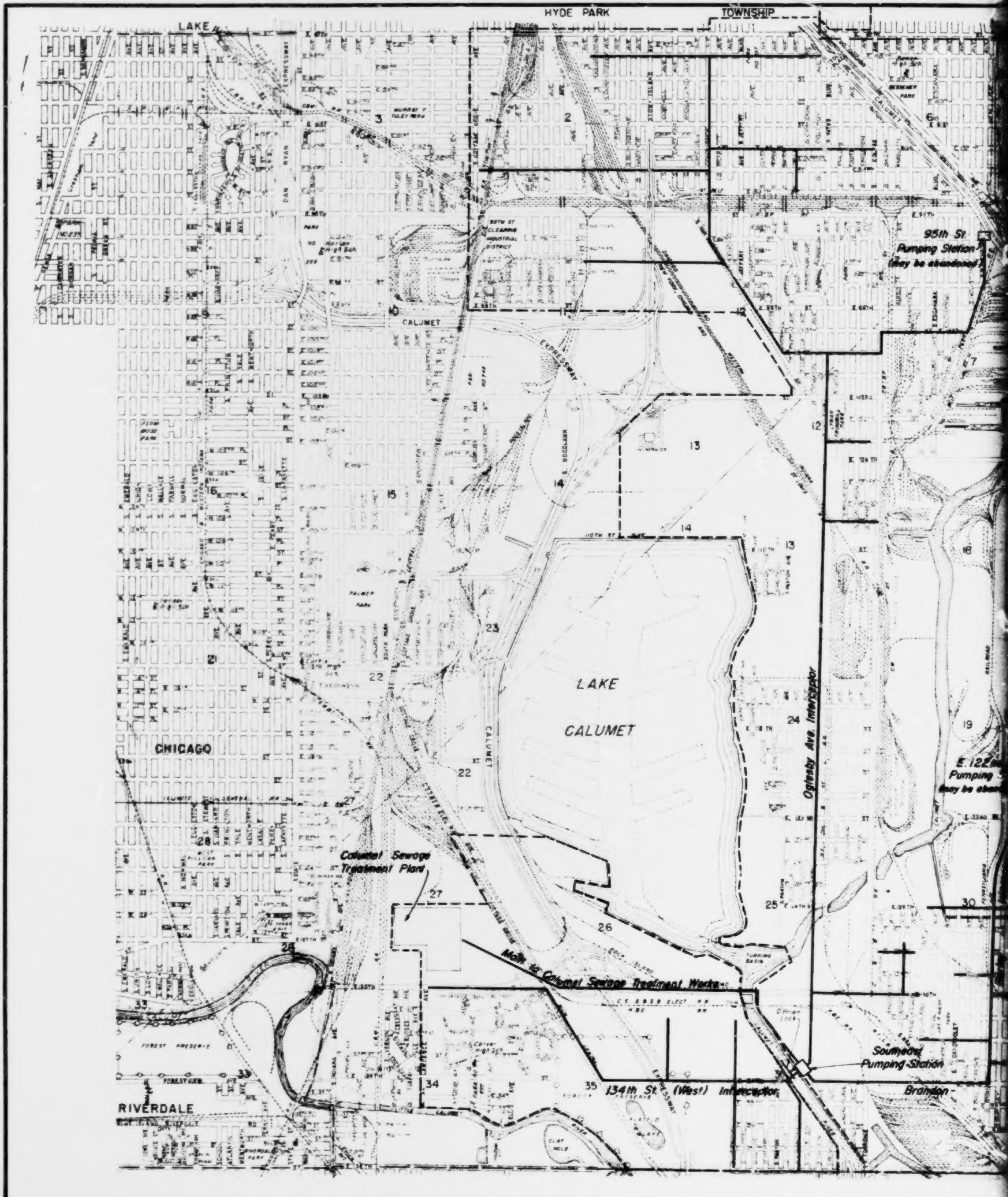
CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

CONVENTIONAL PLAN

(210 Square Miles Service Area)

HARZA ENGINEERING COMPANY DWG.NO. 367 P
BAUER ENGINEERING, INC. PROJECT 6524

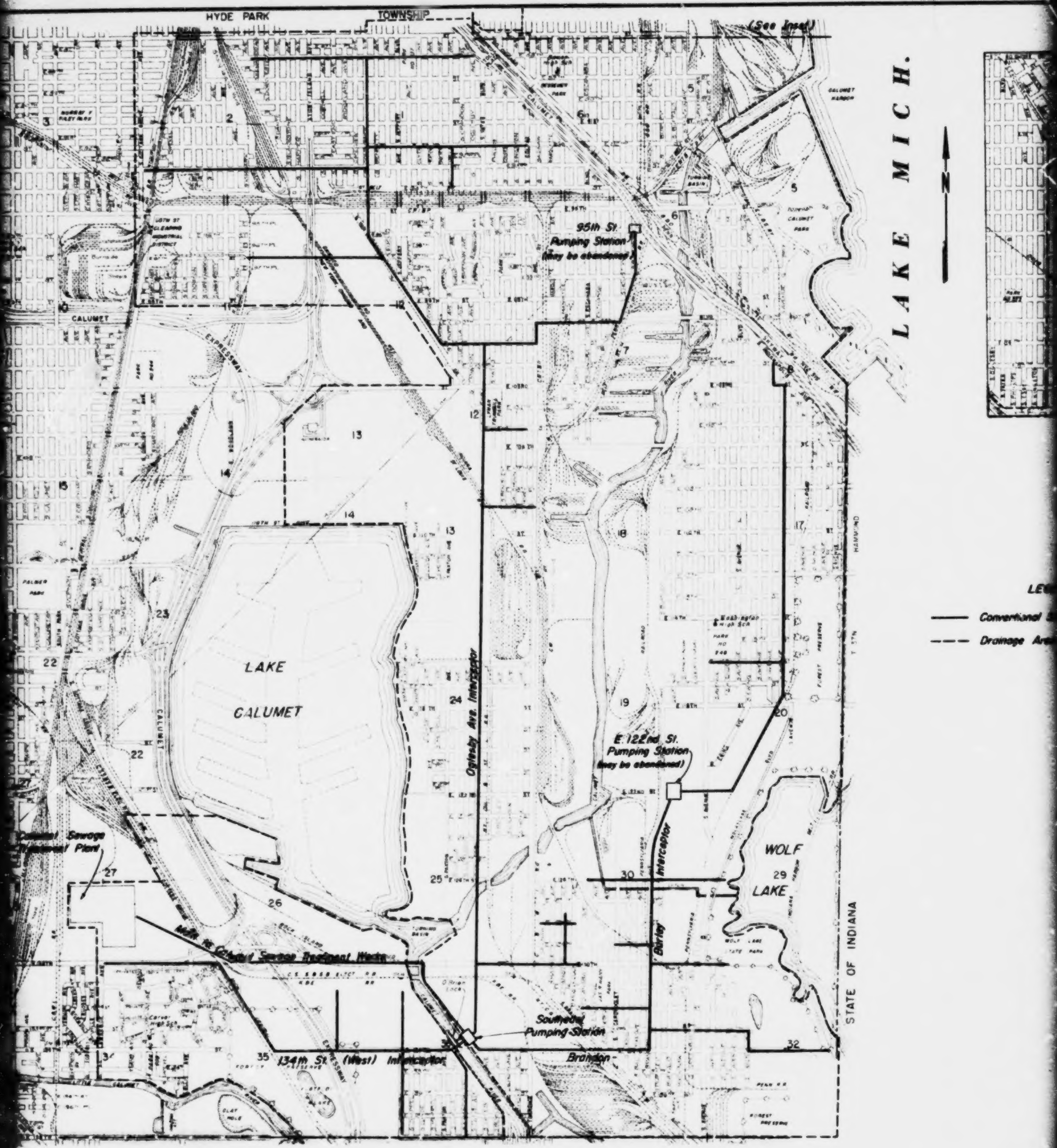
MAP DEVELOPED FROM TOWNSHIP MAPS
PREPARED BY
COOK COUNTY
HIGHWAY DEPARTMENT
CHICAGO, ILLINOIS


HYDE PARK
TOWNSHIP
LAKE
95th St. Pumping Station (may be abandoned)
CALUMET
EXPRESSWAY
LAKE CALUMET
CHICAGO
Oglesby Ave. Interceptor
E. 122
Pumping S
(may be aban
Calumet Sewage Treatment Plant
Main to Calumet Sewage Treatment Works
Southeast Pumping Station
134th St. (West) Interceptor
Brandon
RIVERDALE
FOREST PRESERVE
PALMER PARK
2
3
6
7
9
10
11
12
13
14
15
16
18
19
21
22
23
24
25
26
27
28
30
33
34
35

HARZA ENGINEERING COMPANY DWG. NO. 387 P

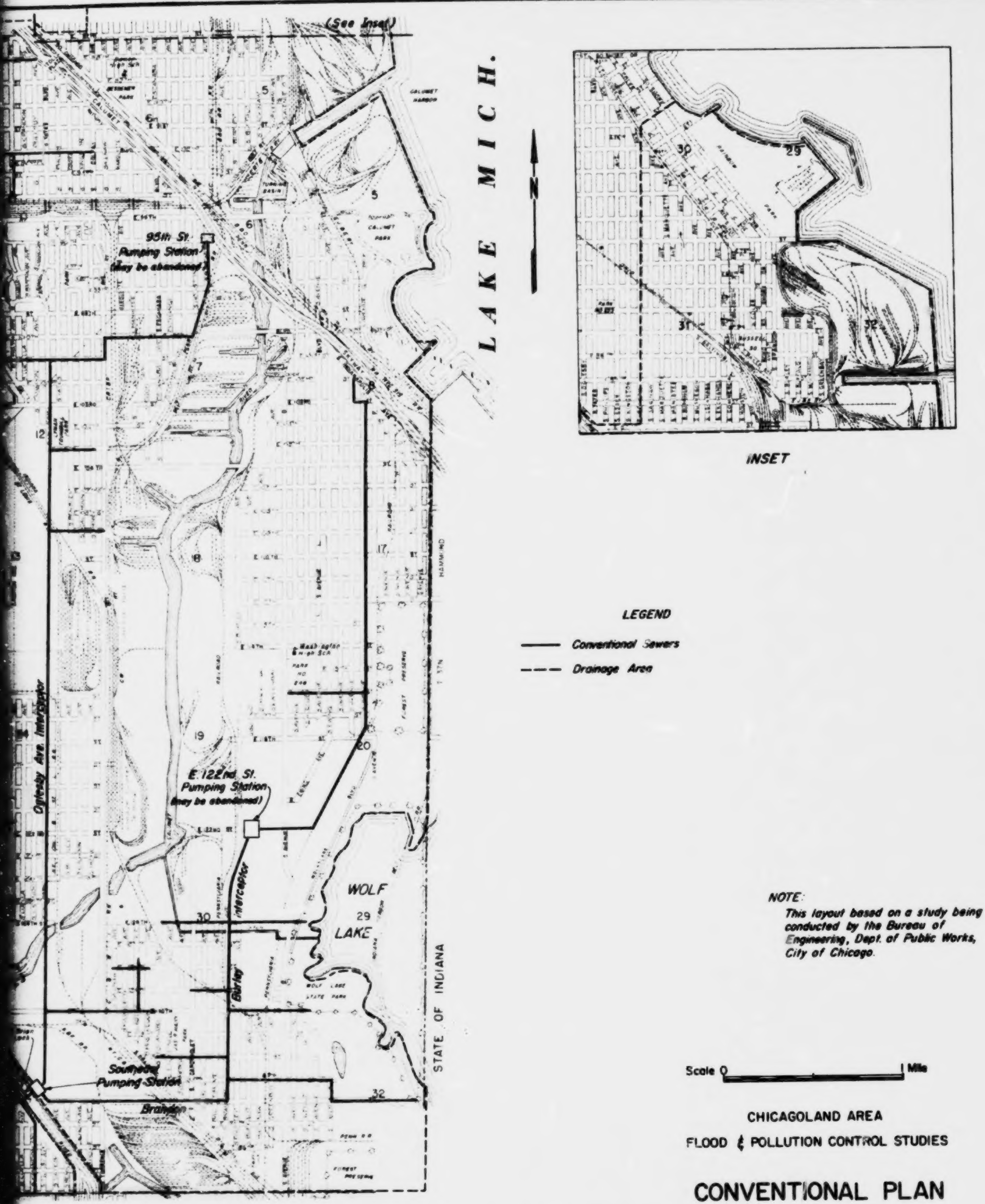
BAUER ENGINEERING, INC. PROJECT 6524

MAP DEVELOPED FROM TOWNSHIP MAPS
PREPARED BY
COOK COUNTY
HIGHWAY DEPARTMENT
CHICAGO, ILLINOIS


HYDE PARK
TOWNSHIP
(See Inset)
LAKE MICH.
CALUMET HARBOR
95th St. Pumping Station (may be abandoned)
CALUMET
EXPRESSWAY
PALMER PARK
LAKE CALUMET
Ogelsby Ave Interceptor
E. 122nd St. Pumping Station (may be abandoned)
WOLF LAKE
WOLF LAKE STATE PARK
Burley Interceptor
Calumet Sewage Treatment Plant
Metr. Sn. Calumet Sewage Treatment Works
Southeast Pumping Station
134th St. (West) Interceptor
Brandon
STATE OF INDIANA
HAMMOND
FOREST PRESERVE
CLAY HOLE
LEG
Conventional S
Drainage Are
1
2
3
5
6
7
8
10
12
13
14
15
17
18
19
20
22
23
24
25
26
27
29
30
31
32
35

MAP DEVELOPED FROM TOWNSHIP MAPS
PREPARED BY
COOK COUNTY
HIGHWAY DEPARTMENT
CHICAGO, ILLINOIS

EXHIBIT 6


(See Inset)
LAKE MICH.
N
CALUMET HARBOR
CALUMET PARK
95th St. Pumping Station (may be abandoned)
E. 122nd St. Pumping Station (may be abandoned)
Oglesby Ave. Interceptor
Interceptor
Burley
Southeast Pumping Station
Brandon
WOLF LAKE
WOLF LAKE STATE PARK
STATE OF INDIANA
HAMMOND
5
6
7
8
12
17
18
19
20
29
30
32
INSET
30
29
31
32

INSET

LEGEND

—— *Conventional Sewers*

– – – *Drainage Area*

NOTE:
This layout based on a study being conducted by the Bureau of Engineering, Dept. of Public Works, City of Chicago.

Scale 0 —— 1 Mile

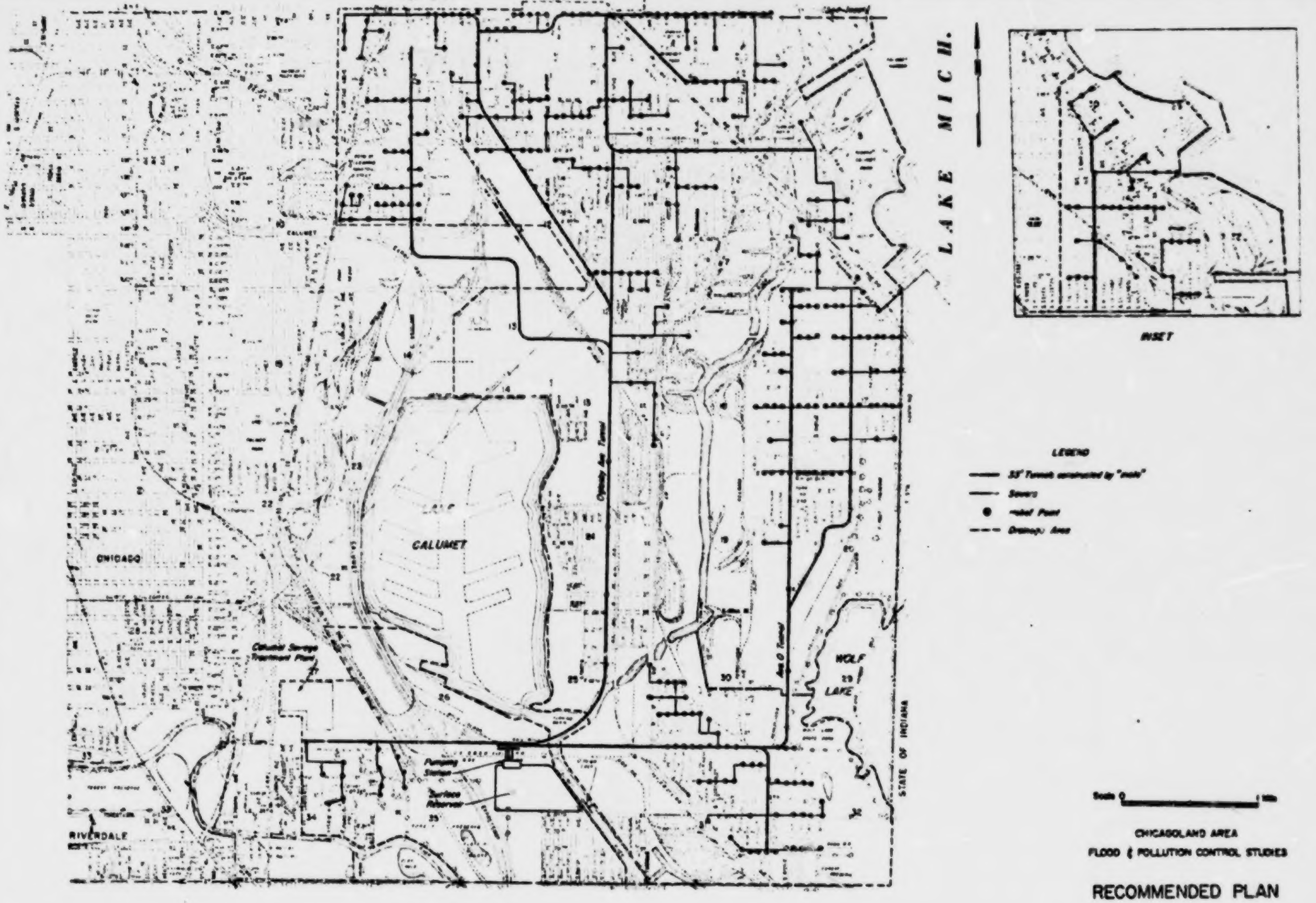
CHICAGOLAND AREA

FLOOD & POLLUTION CONTROL STUDIES

CONVENTIONAL PLAN

(21.0 Square Miles Service Area)

EXHIBIT 7


LAKE MICH.
INSET
CALUMET
CHICAGO
RIVERDALE
WOLF LAKE
STATE OF INDIANA
Calumet Sewage Treatment Plant
Pumping Station
Surface Reservoir
LEGEND
33' Tunnels constructed by "mole"
Sewers
Inlet Point
Drainage Area

Scale 0 — 1 Mile

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

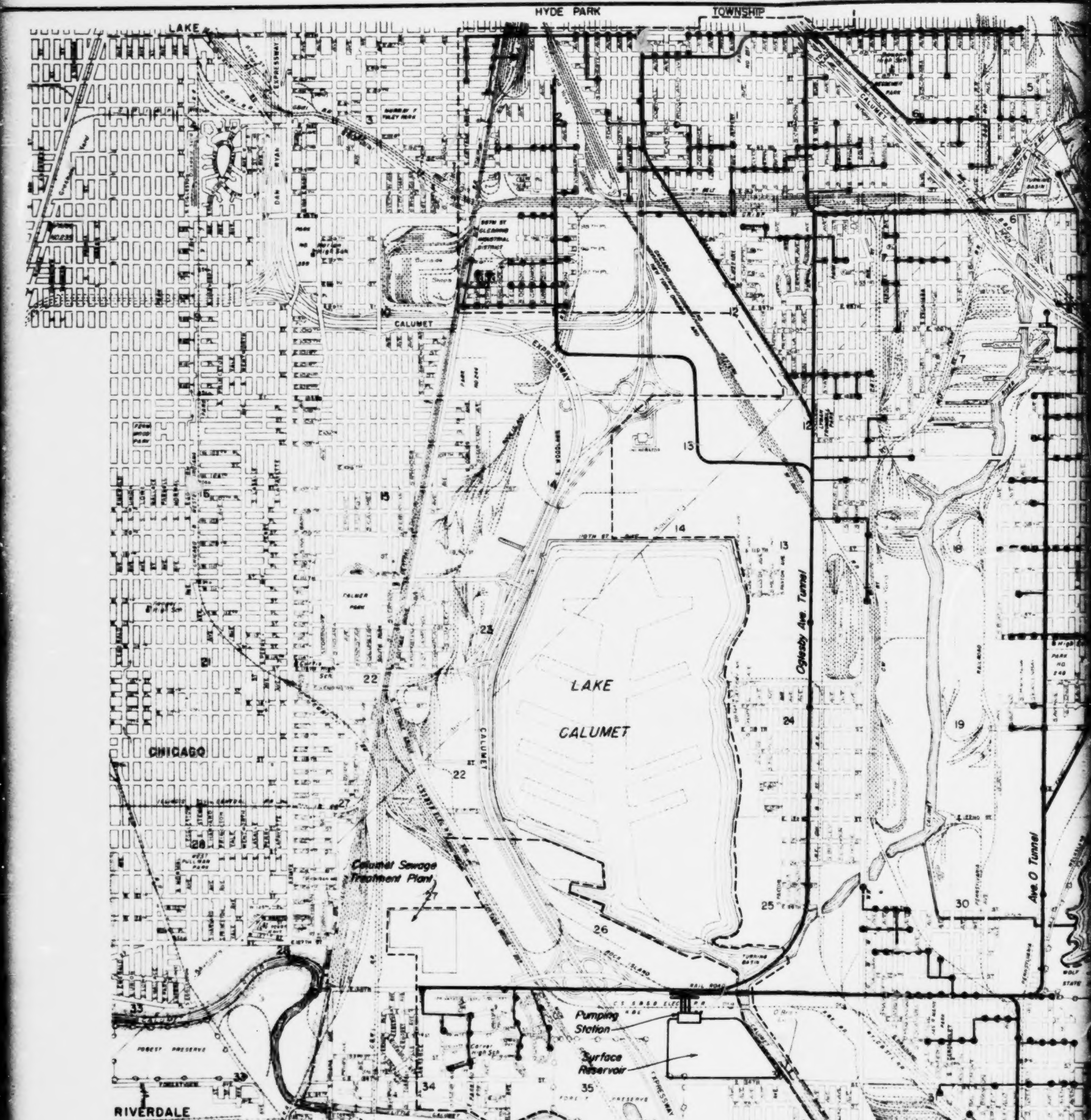
RECOMMENDED PLAN

(Plan IC-210 Square Miles Service Area)

HARZA ENGINEERING COMPANY DWG NO. 387 P
BAUER ENGINEERING, INC PROJECT 6524

MAP DEVELOPED FROM TOPOGRAPHIC MAPS
PREPARED BY
COOK COUNTY
HIGHWAY DEPARTMENT
CHICAGO, ILLINOIS

F 110


HYDE PARK TOWNSHIP
LAKE
CALUMET
EXPRESSWAY
LAKE CALUMET
CHICAGO
RIVERDALE
Oglesby Ave. Tunnel
Ave. O Tunnel
Calumet Sewage Treatment Plant
Pumping Station
Surface Reservoir
FOREST PRESERVE
TURNING BASIN
ROCK ISLAND RAIL ROAD
TALMAN PARK
WEST PULLMAN PARK
CLAY HOLE
INCINERATOR
2
3
5
6
7
10
12
13
14
15
16
18
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33
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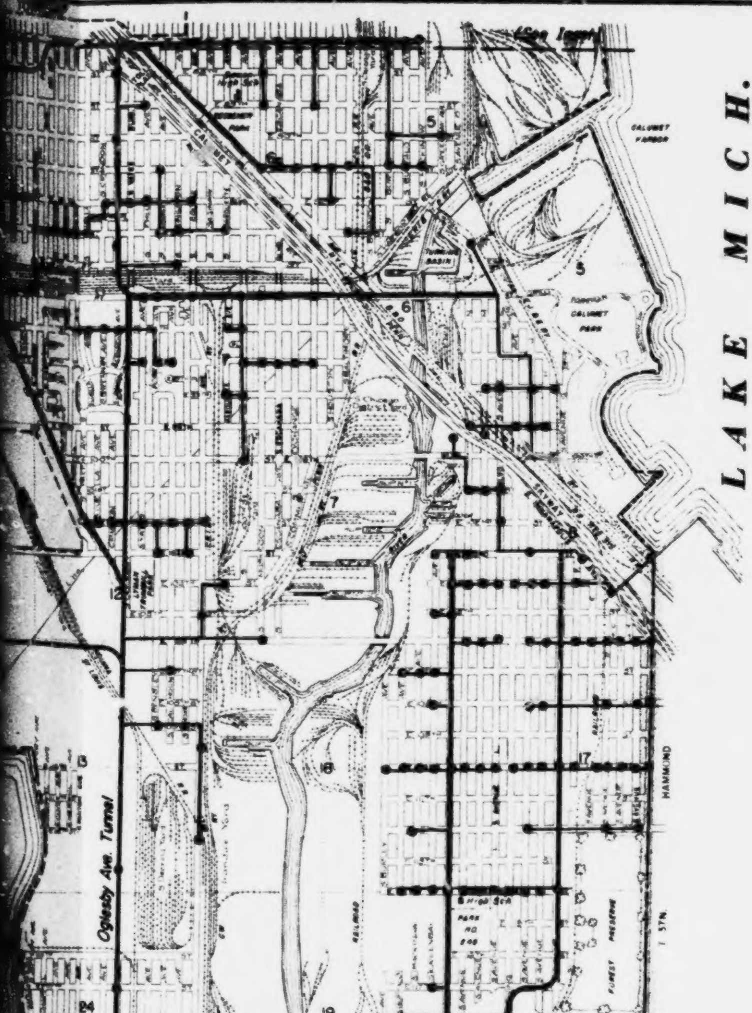
MAP DEVELOPED FROM TOWNSHIP MAPS

EXHIBIT 7


(See Inset)
LAKE MICH.
Calumet Harbor
Calumet Park
Oglesby Ave. Tunnel
Ave. O Tunnel
WOLF LAKE
Wolf Lake State Park
Forest Preserve
Hammond
STATE OF INDIANA
Penn R.R.
5
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7
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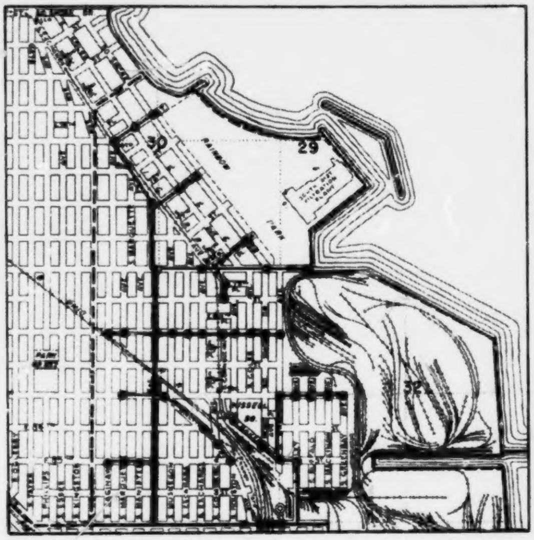
32



INSET

LEGEND

- *33' Tunnels constructed by "mole"*
- *Sewers*
- ● *Relief Point*
- *Drainage Area*

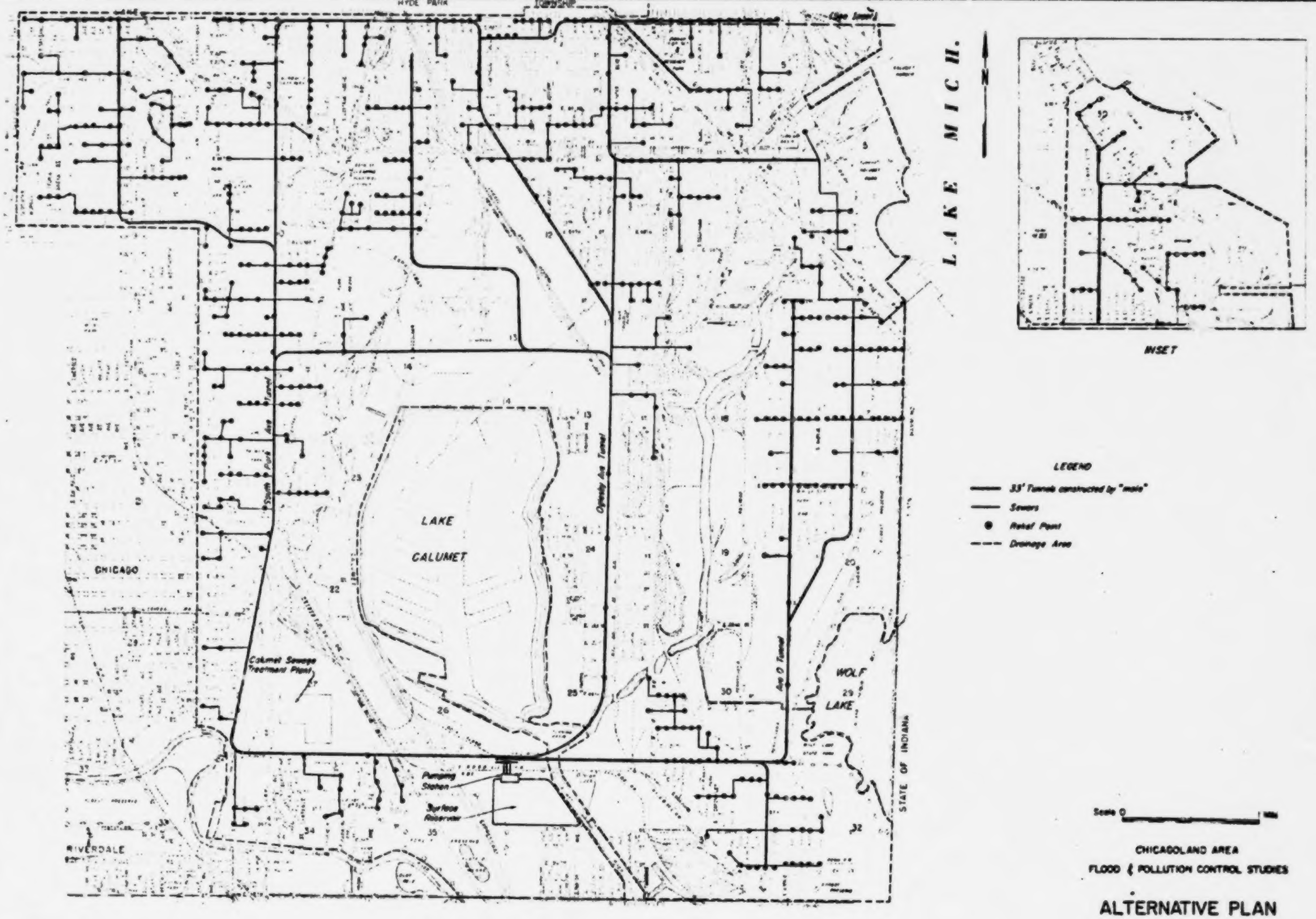
Scale 0 — 1 Mile

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

RECOMMENDED PLAN

(Plan IC-21.0 Square Miles Service Area)

EXHIBIT 8



HYDE PARK
TOWNSHIP
(See Inset)
LAKE MICH.
N
INSET
LAKE CALUMET
WOLF LAKE
STATE OF INDIANA
CHICAGO
RIVERDALE
Calumet Sewage Treatment Plant
Pumping Station
Surface Reservoir
South Park Ave Tunnel
Ogelsby Ave Tunnel
Ave O Tunnel
LEGEND
33' Tunnels constructed by "mole"
Sewers
Relief Point
Drainage Area
Scale 0 1 Mile

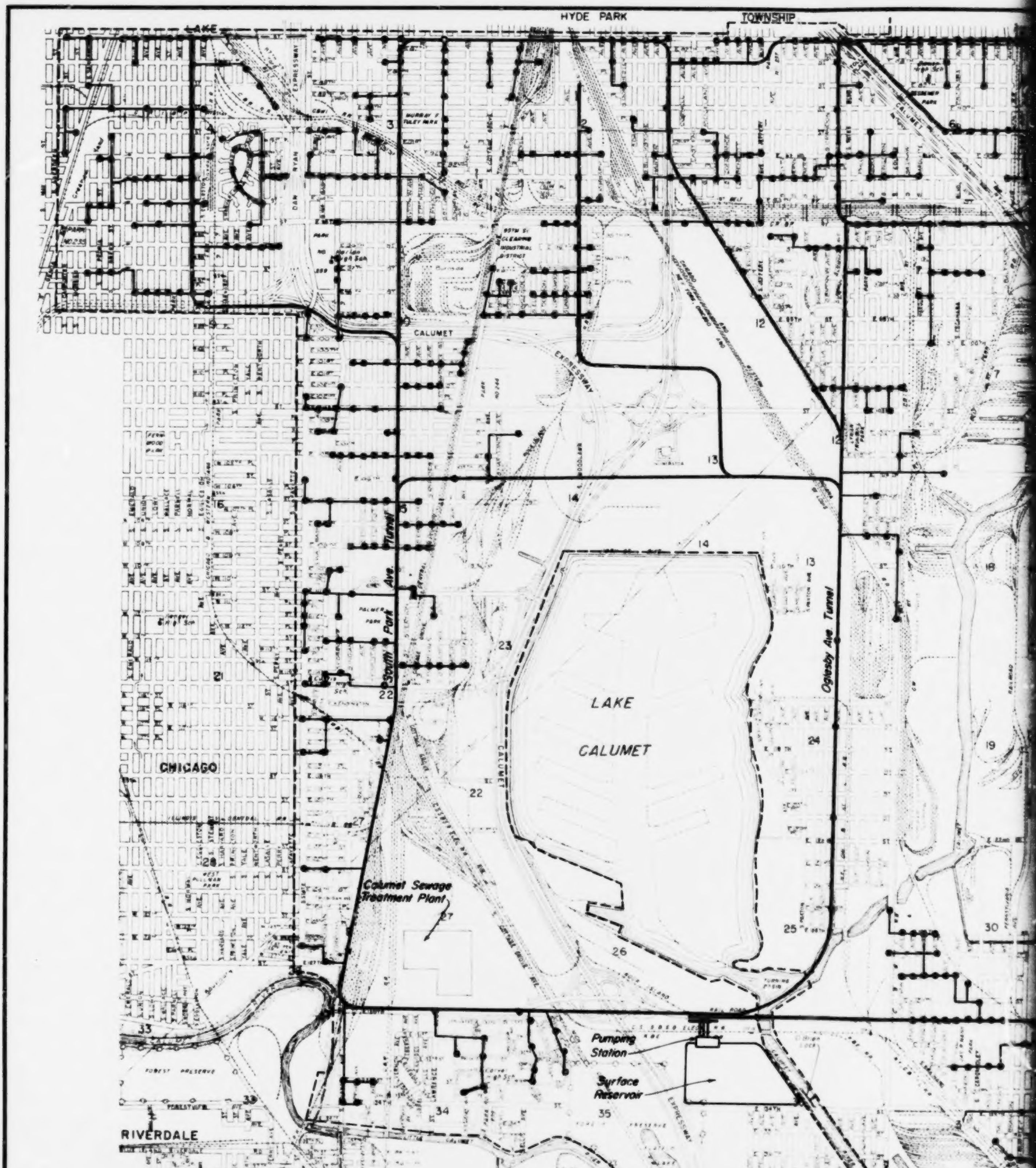
CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

ALTERNATIVE PLAN

(Plan 2A - 29.7 Square Miles Service Area)

HARZA ENGINEERING COMPANY DWG NO 387 P
BAUER ENGINEERING, INC PROJECT 6524

COOK COUNTY
HIGHWAY DEPARTMENT



HYDE PARK
TOWNSHIP
LAKE
CALUMET
EXPRESSWAY
DAN RYAN EXPRESSWAY
CALUMET
South Park Ave. Tunnel
Oglesby Ave. Tunnel
LAKE
CALUMET
CHICAGO
Calumet Sewage Treatment Plant
Pumping Station
Surface Reservoir
RIVERDALE
FOREST PRESERVE
PALMER PARK
RIVERDALE PARK
3
2
1
6
12
13
14
15
16
18
19
22
23
24
25
26
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28
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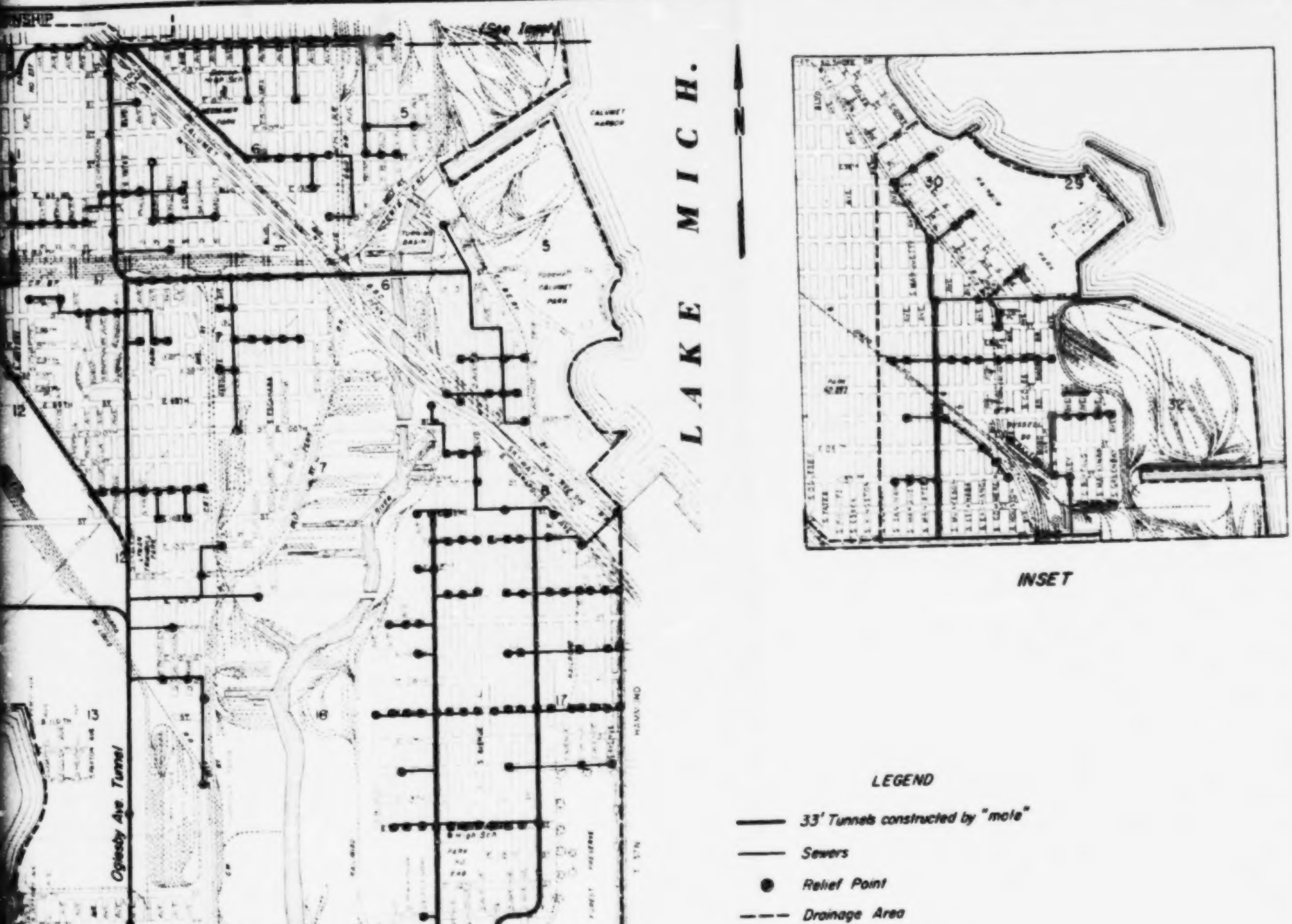
33
34
35

HARZA ENGINEERING COMPANY DWG. NO. 367 P

MAP DEVELOPED FROM TOWNSHIP MAPS

EXHIBIT 8


(See Inset)
LAKE MICH.
N
CALUMET
5
6
7
12
13
17
18
19
20
24
25
29
30
32
Oglesby Ave. Tunnel
Ave. O Tunnel
WOLF
LAKE
STATE OF INDIANA
CALUMET HARBOR
CALUMET PARK
TURNING BASIN
WOLF LAKE STATE PARK
FOREST PRESERVE

INSET

LEGEND

- ▬ 33' Tunnels constructed by "mole"
- — Sewers
- ● Relief Point
- - - - Drainage Area

Scale 0 ——— 1 Mile

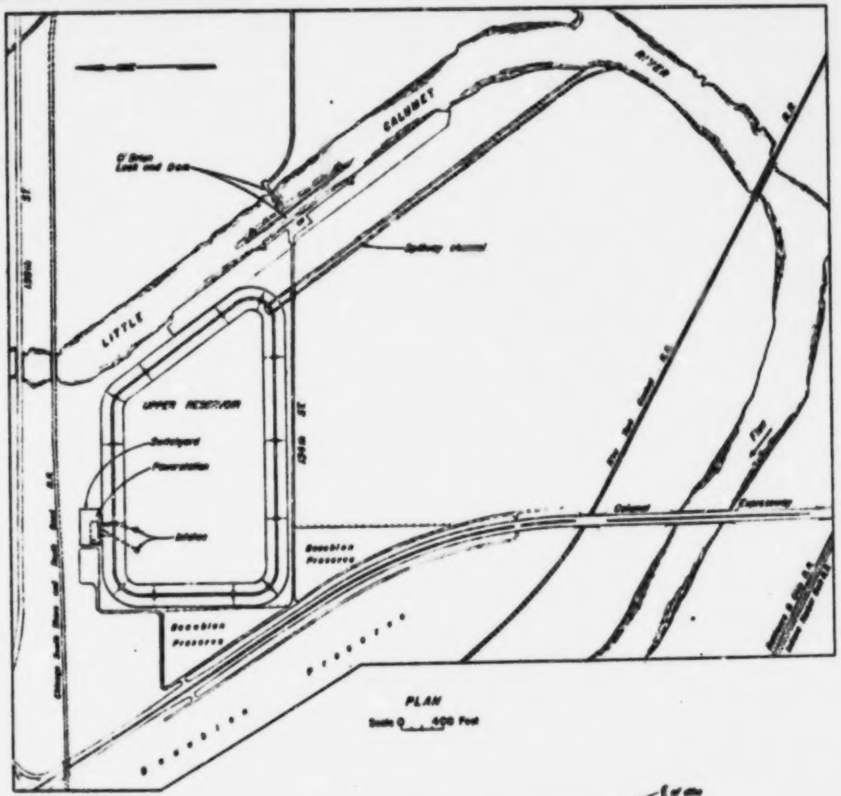
CHICAGOLAND AREA

FLOOD & POLLUTION CONTROL STUDIES

ALTERNATIVE PLAN

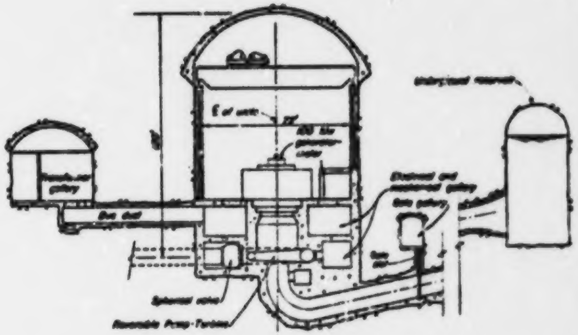
(Plan 2A - 29.7 Square Miles Service Area)

EXHIBIT 9


O'Brien Lock and Dam
CALUMET
LITTLE
RIVER
Spillway channel
UPPER RESERVOIR
Switchyard
Powerstation
Intakes
Beaubien Preserve
Beaubien Preserve
Calumet Expressway
130th ST
134th ST

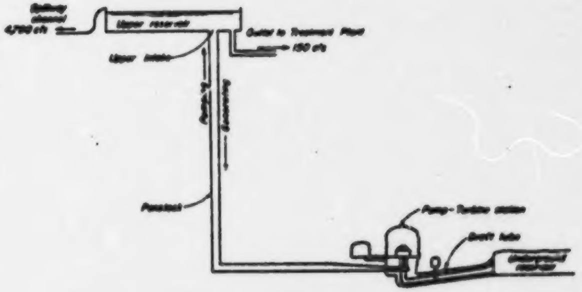
PLAN

Scale 0 ___ 400 Feet


Underground reservoir
℄ of units
Transformer gallery
Bus duct
Electrical and mechanical gallery
Gate gallery
Spherical valve
Reversible Pump-Turbine

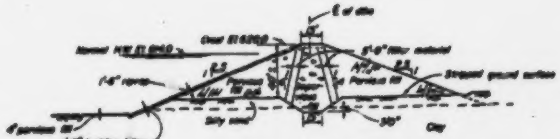
PUMP-TURBINE STATION

Scale 0 ___ 30 Feet


Spillway channel
4700 cfs
Upper reservoir
Outlet to Treatment Plant
150 cfs
Upper intake
Pumping
Generating
Penstock
Pump-Turbine station
Draft tube
Underground reservoir

SCHEMATIC LAYOUT OF REVERSIBLE PUMP-TURBINE STATION

N.T.S.


℄ of dike
Crest El 620.0
Normal HW El 610.0
5'-0" filter material
Stripped ground surface
1'-6" riprap
Pervious fill
Silty sand
Clay
4' pervious fill
1'-0" bedding filter
30"

RESERVOIR DIKE SECTION

Scale 0 ___ 30 Feet

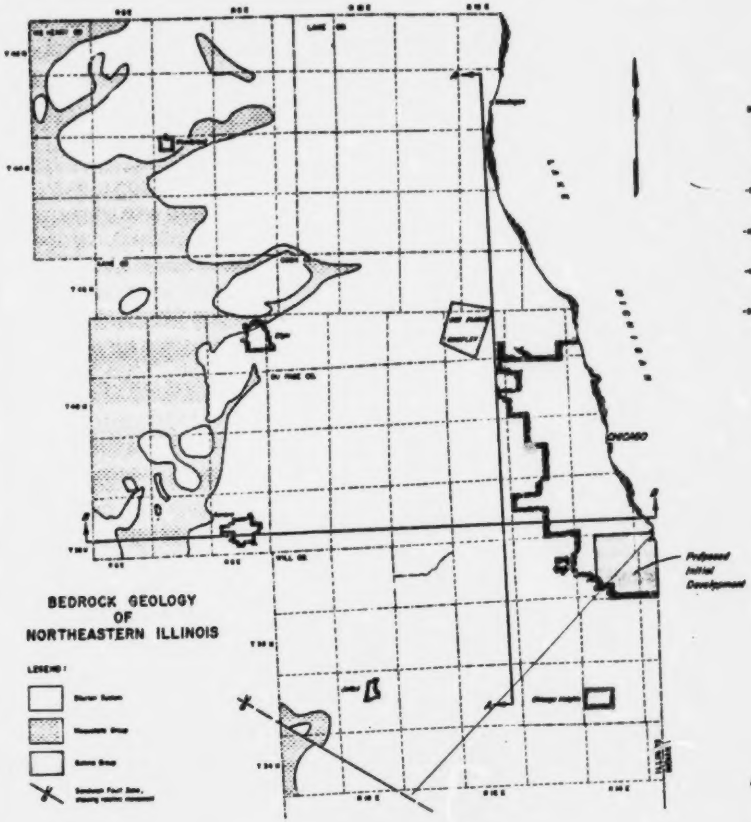
CHICAGOLAND AREA

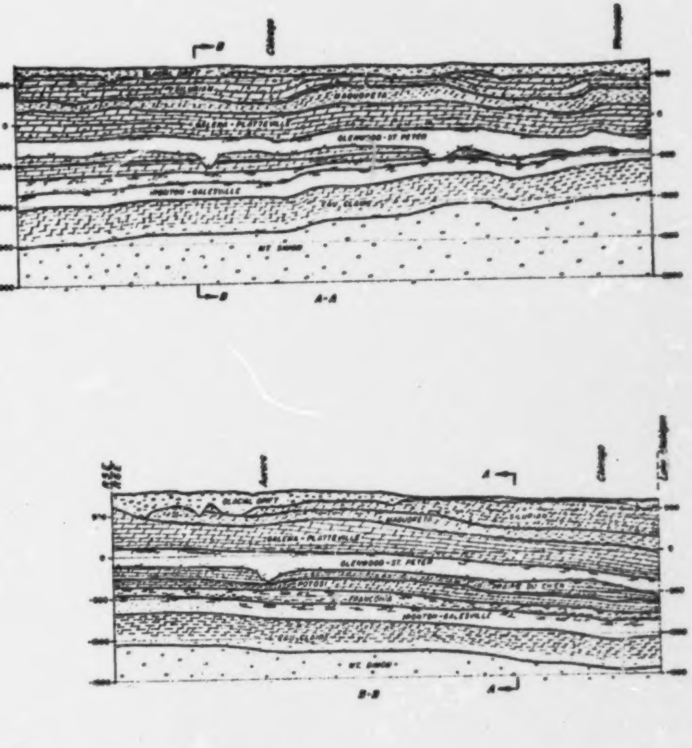
FLOOD & POLLUTION CONTROL STUDIES

SURFACE RESERVOIR
AND PUMP-TURBINE STATION

HARZA ENGINEERING COMPANY DWG NO 387 P-62
BAUER ENGINEERING, INC. PROJECT 6604

EXHIBIT-10


BEDROCK GEOLOGY
OF
NORTHEASTERN ILLINOIS
LEGEND:
Silurian System
Maquoketa Group
Galena Group
Sandwich Fault Zone, showing relative movement
LAKE MICHIGAN
CHICAGO
Proposed Initial Development
A
B
SCALE
MILES


A-A
B-B
MAQUOKETA
GALENA-PLATTEVILLE
GLENWOOD-ST PETER
IRONTON-GALESVILLE
EAU CLAIRE
MT SIMON
GLACIAL DRIFT
SILURIAN
POTOSI
FRANCONIA
PRAIRIE DU CHIEN
0

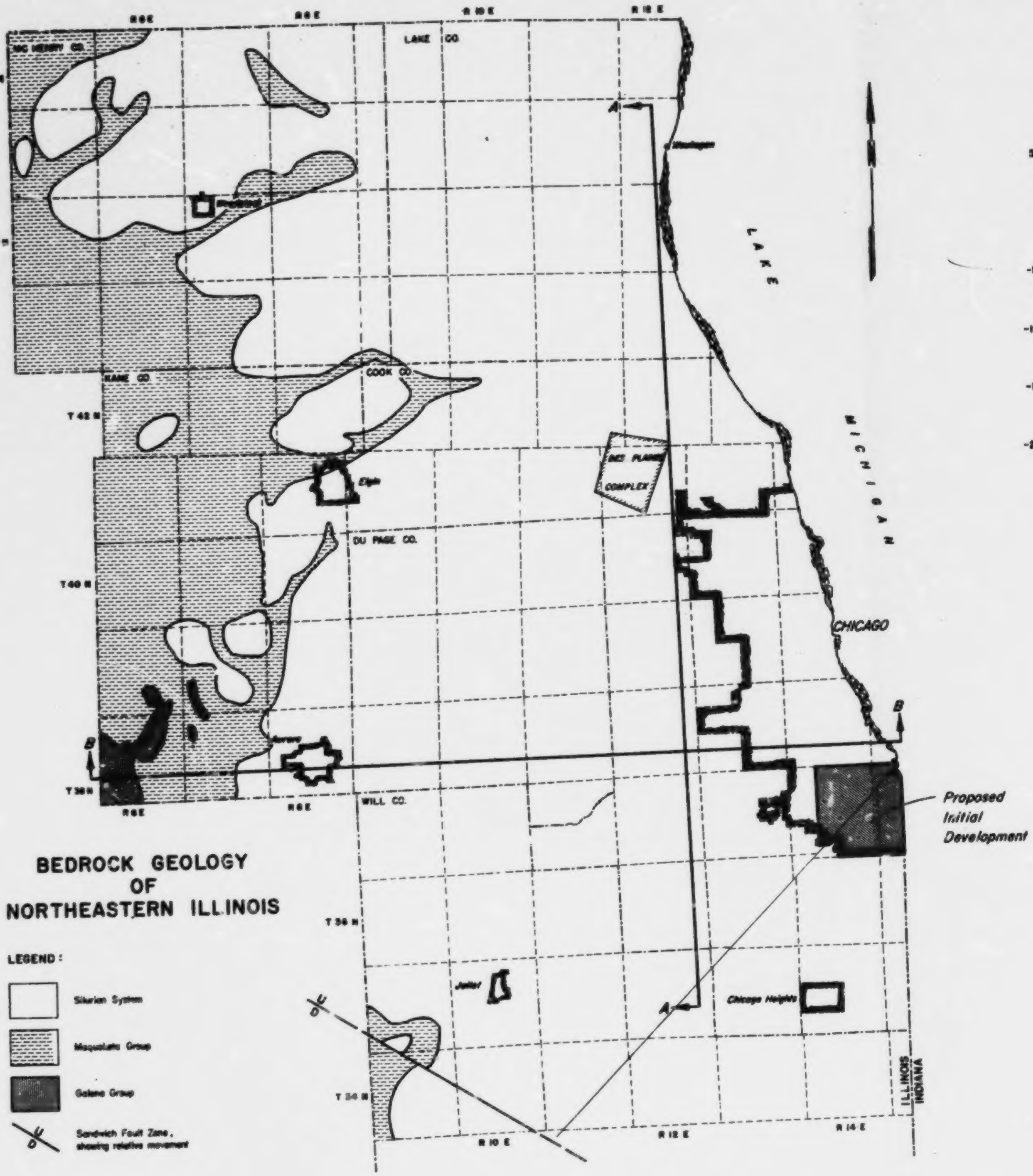
NOTES:

1 Data for geologic map from Cooperative Ground-water Report 1, Preliminary Report on Ground-water Resources of the Chicago Region, Illinois. Illinois State Water Survey, and Illinois State Geological Survey, 1959
2 For descriptions of geologic units and symbols used on sections, see Exhibit D-3, Generalized Geologic Column.
3 Vertical exaggeration of sections approximately 42 x

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

GEOLOGIC MAP AND SECTIONS

HARZA ENGINEERING COMPANY DWG.NO. 387 P-2
BAUER ENGINEERING, INC. PROJECT 5524


R 10 E
R 12 E
MC HENRY CO.
LAKE CO.
KANE CO.
COOK CO.
DU PAGE CO.
WILL CO.
T 42 N
T 40 N
T 36 N
T 34 N
A
A
B
B
Elgin
Joliet
Chicago Heights
DES PLAINES COMPLEX
CHICAGO
LAKE MICHIGAN
N
Proposed Initial Development
ILLINOIS
INDIANA
R 10 E
R 12 E
R 14 E

BEDROCK GEOLOGY
OF
NORTHEASTERN ILLINOIS

LEGEND:

☐	Silurian System
▤	Maquoketa Group
■	Galena Group
U/D	Sandwich Fault Zone, showing relative movement

SCALE 0 5 MILES

500
0
-500
-1000
-1500
-2000
500
0
-500
-1000
-1500

NOTES:

1 Data for geologic map fr
Ground-water Resources
Illinois State Geological

2. For descriptions of geol
Generalized Geologic C

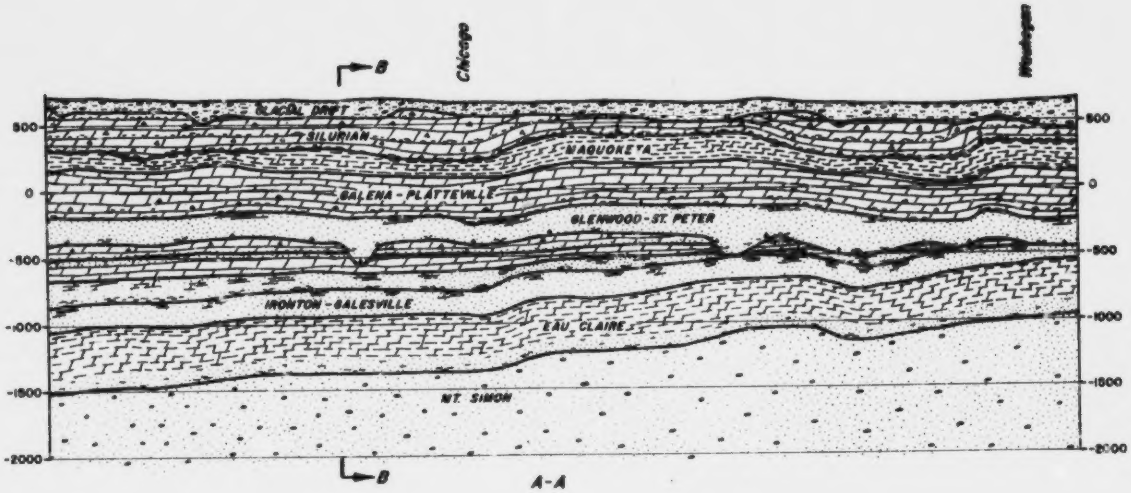
3 Vertical exaggeration of

HARZA ENGINEERING COMPANY DWG. NO. 387 P3

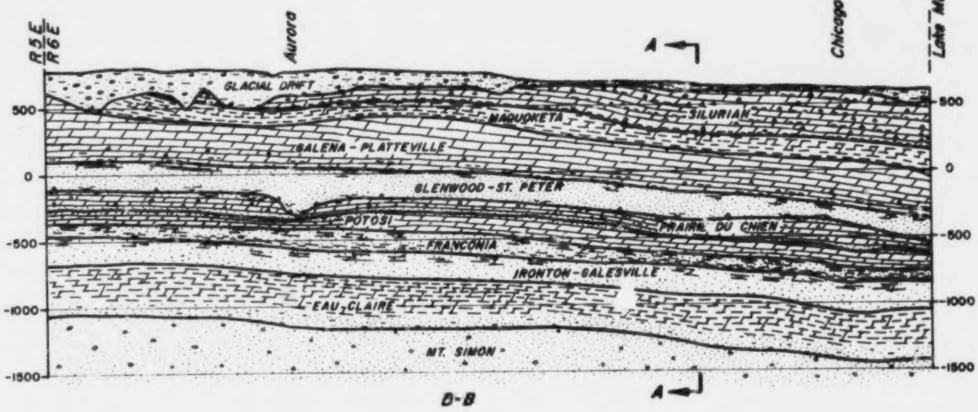
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT -10

MICHIGAN
CHICAGO
B
Proposed Initial Development
ILLINOIS
INDIANA


B
Chicago
Michigan
GLACIAL DRIFT
SILURIAN
MAQUOKETA
GALENA-PLATTEVILLE
GLENWOOD-ST. PETER
IRONTON-GALESVILLE
EAU CLAIRE
MT. SIMON
500
0
-500
-1000
-1500
-2000
B

A-A


R5E
R6E
Aurora
A
Chicago
Lake Michigan
GLACIAL DRIFT
MAQUOKETA
SILURIAN
GALENA-PLATTEVILLE
GLENWOOD-ST. PETER
POTOSI
PRAIRIE DU CHIEN
FRANCONIA
IRONTON-GALESVILLE
EAU CLAIRE
MT. SIMON
500
0
-500
-1000
-1500
A

B-B

NOTES:

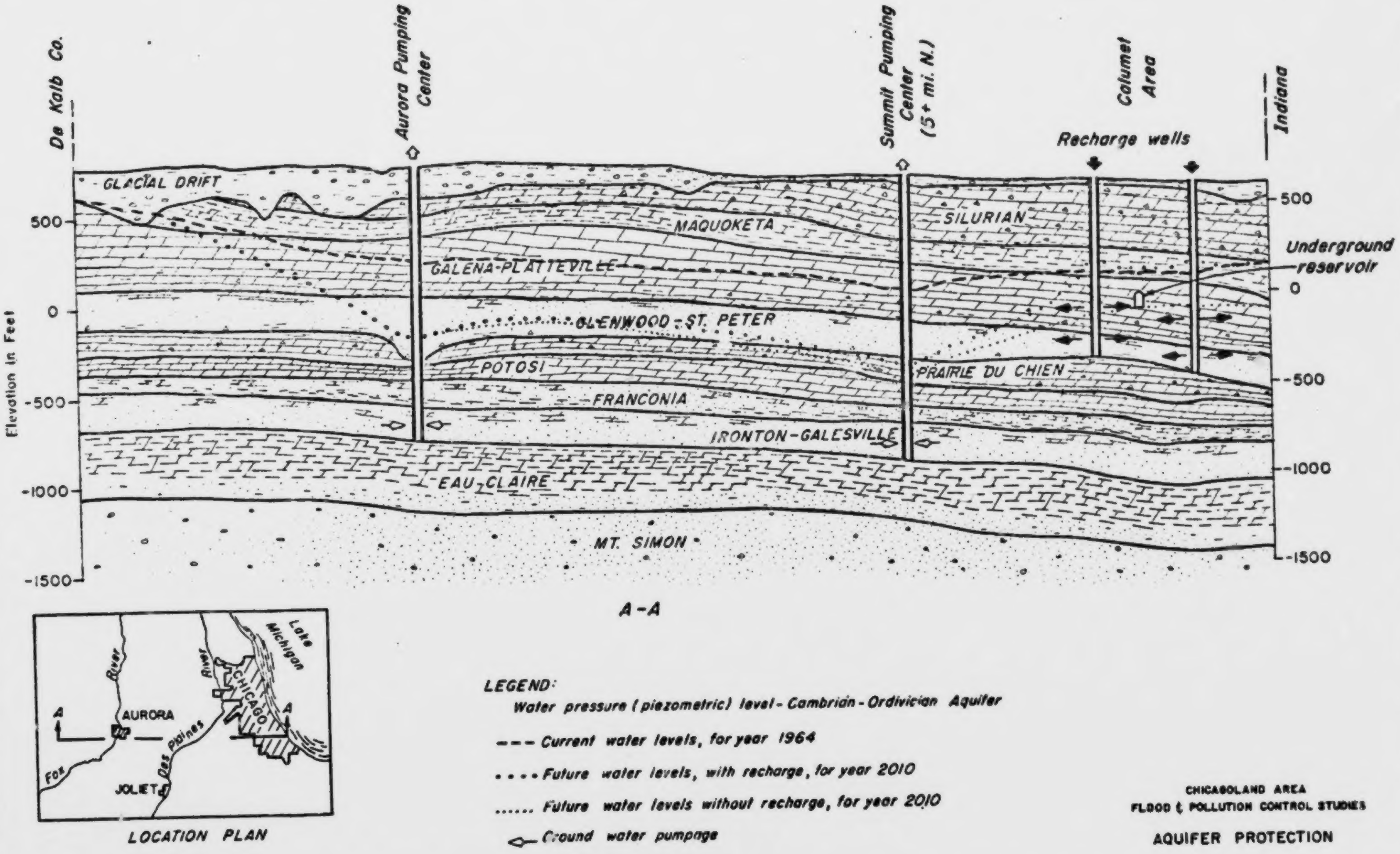
1. Data for geologic map from Cooperative Ground-water Report 1, Preliminary Report on Ground-water Resources of the Chicago Region, Illinois. Illinois State Water Survey, and Illinois State Geological Survey, 1959
2. For descriptions of geologic units and symbols used on sections, see Exhibit B-3, Generalized Geologic Column
3. Vertical exaggeration of sections approximately 42 x.

CHICAGOLAND AREA

FLOOD & POLLUTION CONTROL STUDIES

GEOLOGIC MAP AND SECTIONS

EXHIBIT-11


De Kalb Co.
Aurora Pumping Center
Summit Pumping Center (5+ mi. N.)
Calumet Area
Recharge wells
Indiana
Underground reservoir
Elevation in Feet
500
0
-500
-1000
-1500
GLACIAL DRIFT
MAQUOKETA
SILURIAN
GALENA-PLATTEVILLE
GLENWOOD - ST. PETER
POTOSI
PRAIRIE DU CHIEN
FRANCONIA
IRONTON-GALESVILLE
EAU CLAIRE
MT. SIMON
A-A
Lake Michigan
River
AURORA
CHICAGO
A
Fox
JOLIET
Des Plaines
LOCATION PLAN

LEGEND:

Water pressure (piezometric) level - Cambrian - Ordivician Aquifer

- - - Current water levels, for year 1964

• • • • Future water levels, with recharge, for year 2010

...... Future water levels without recharge, for year 2010

⇦ Ground water pumpage

◀ Ground water recharge

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

AQUIFER PROTECTION

HARZA ENGINEERING COMPANY DWG. NO. 387P-4
BAUER ENGINEERING, INC. PROJECT 6524

APPENDICES

A. Hydrology and Hydraulics

B. Geology

C. Water Quality Considerations

D. Aquifer Protection

E. Power

F. Construction Methods and Costs

G. Benefits from Flood and Pollution Control

Appendix A

HYDROLOGY AND HYDRAULICS

Analytical Procedures
Computed Runoff from Historical Storms
Hydraulic Characteristics of Existing Systems
Hydraulic Characteristics of Deep Tunnel Plan
Hydrologic Evaluation of Deep Tunnel Plan
Questionnaire Survey
Future Investigations

Exhibits

Appendix A

HYDROLOGY AND HYDRAULICS

Analytical Procedures

Exhibit A-1 defines the relationships between rainfall intensity, duration, and frequency of occurrence which were used in this study. These are the same relationships as used by the Bureau of Engineering, City of Chicago, from whom the chart was obtained.

Infiltration

Exhibit A-2 defines the relationships used in this study for infiltration into pervious areas. Mr. Clint Keifer of the Bureau of Engineering, City of Chicago, has made possible this verification of the relationship shown through establishment of a 14-acre experimental watershed in a fully built up part of the city. Simultaneous measurements of rainfall and runoff from the watershed have shown that the relationship of Exhibit A-2 is a useful interpretation of the infiltration into Chicago soils.

Other estimates of infiltration capacity of Chicago soils have been made prior to the Deep Tunnel investigations in storm-drainage studies of two Chicago suburbs by Bauer Engineering, Inc. In those studies a detailed accounting was made of three components of storm water that stored temporarily on the watershed, that discharged through the sewers, and that which infiltrated into the ground. House-to-house surveys were made to measure volumes ponded on the streets in low places. Computations were made on an hour-by-hour basis through storm periods to account for the distribution of the volume of storm water which resulted. For pervious areas, it was found that a satisfactory accounting could be attained by adopting an initial loss equivalent to the cumulative rainfall amount, up to a rainfall accumulation of 1.0 inch, followed by a uniform infiltration loss of 0.25 inches per hour. This relationship is shown as a dashed line on Exhibit A-2.

Part of the infiltration loss occurs after the storm has ceased, the water infiltrating from depression storage and overland flow. Thus the period of opportunity for infiltration is longer than the actual duration of the storm.

Perviousness

For purposes of study of the Deep Tunnel Plan, the Calumet area was divided into two parts: (a) that portion presently or potentially draining into sewers, and (b) the remainder, consisting of forest preserves, rivers, lakes, and similar areas. It was assumed that 36 percent of the first portion is presently or potentially covered with roofs and pavements directly connected to sewers.

Table A-1 summarizes the types of land use in the Calumet area.

Neighborhood Storage

Neighborhood storage is defined as the sum of the storage volumes in the following places:

(a) Lateral sewers flowing full

(b) Street inlets at peak discharge

(d) House sewers, gutters, downspouts and drain tile flowing full

Table A-1

LAND USE IN CALUMET AREA

	Area Served	
Land use	Plan 1C	Plan 2A
Residential and commercial	4,880 acres	7,380 acres
Industrial, highways, railroads	3,700	5,500
Public uses connected to sewers (schools, small parks)	770	1,120
Vacant lands potentially connectable to sewers (new or existing)	2,700	3,700
Future filling of Lake Calumet	220	450
Subtotal	12,270 acres	18,150 acres
Public lands not connected to sewers (forest preserve, incinerator, sewage treatment works	800	1,550
Water areas	720	1,670
Totals	13,790 acres	21,370 acres

(e) Overland flow over lawns, walks, and alleys

(f) Depression storage on lawns, gardens, and pavements

(g) Basement storage

(h) Viaduct storage

(i) Leakage through sewer joints

All of these volumes have an influence on storm drainage. The last three are usually undesirable and should be minimized. Table A-2 presents an estimate of the magnitudes of these storage volumes. It can be noted that about 0.25 inch of neighborhood storage is acceptable. This amount also is necessary to the normal functioning of the various storm water conductors and to obtain the overland flow associated with severe storms.

Hypothetical Storm Computations

The curves of Exhibit A-1 were translated into hypothetical storms for the purpose of comparing drainage systems. Each curve of the exhibit was taken to be a time variation of rainfall intensity for a particular storm. A series of computations was made to determine the volume of neighborhood storage required for each hypothetical storm, for several assumed magnitudes of sewer outflow capacity.

A sample computation is presented in Table A-3. The average rainfall intensities and cumulative amounts of rainfall are shown for each of several time periods during a given storm. The portion of the rain falling on the impervious area (paved or roofed) is assumed to run off directly, less 0.0625 inches of depression storage. The portion of the rain falling on the remaining 64 percent of the area which is pervious, must first satisfy the infiltration capacity and the assumed

Table A-2

NEIGHBORHOOD STORAGE VOLUMES

(Typical 10-Acre Watershed)

Item	Length or Number (Feet)	Area (Square Feet)	Volume (Cubic Feet)	Volume (Inches)
Lateral sewer	1320	1.0	1320	0.0365
Street inlets	12	8.0	200	0.0055
Gutters	3200	0.3	960	0.0265
House sewers	3000	0.08	240	0.0066
House drain tile	6000	0.08	480	0.0133
House gutters	6000	0.04	240	0.0066
Overland flow	-	-	-	0.1500
Subtotal				0.2450
Depression storage (0.64 x 0.25 + 0.36 x 0.06)				0.1800
Basement storage				(See Exhibit A-7)
Viaduct storage				not estimated
Leakage through sewer joints				not estimated [1]

---

[1] Storage in soil surrounding the sewer may be substantial with a leaky sewer passing through permeable soil such as sand. However, it is doubtful if this volume is greater than 0.1 inch in the study areas.

Table A-3

SAMPLE COMPUTATIONS OF RUNOFF FROM RAINFALL

(100-year rainfall, fully advanced)

Cumulative Time (Min.) (1)	Average Rainfall (Inches per Hour (2)	Cumulative Rainfall (Inches) (3)	Losses on Pervious Area (Inches) (4)	Pervious Runoff (Inches) (5)	Impervious Runoff (Inches) (6)	36% of Col. (6) (Inches) (7)	64% of Col. (5) (Inches) (8)	Runoff Col. (7) plus Col. (8) (Inches) (9)	$\frac{\text{Col. (1)}}{60}$ times 1.0/hr. (Inches) (10)	Col. (9) minus Col. (10) (Inches) (11)
5	9.6	0.80	0.45	0.35	0.74	0.27	0.22	0.49	0.083	0.407
10	7.75	1.29	0.62	0.67	1.23	0.44	0.44	0.88	0.16	0.72
15	6.6	1.65	0.80	0.85	1.59	0.57	0.54	1.11	0.25	0.86
20	5.8	1.93	0.87	1.06	1.87	0.68	0.68	1.36	0.33	1.03
40	3.9	2.60	1.15	1.45	2.54	0.91	0.93	1.84	0.67	1.17
60	2.95	2.95	1.37	1.58	2.89	1.04	1.01	2.05	1.0	1.05
80	2.42	3.22	1.55	1.67	3.16	1.14	1.07	2.21	1.33	0.88
100	2.06	3.42	1.70	1.75	3.39	1.24	1.12	2.36	1.67	0.69
120	1.8	3.60	1.90	1.70	3.54	1.27	1.09	2.36	2.0	0.36
160	1.46	3.89	2.25	1.57	3.76	1.35	1.01	2.36	2.67	
240	1.06	4.24	2.65	1.59	4.18	1.50	1.02	2.52	4.0	

References:
(a) DC-56, Chicago Bureau of Engineering Rainfall Intensity Curves
(b) Infiltration Curve of p. 56, Mar. '59 Journal San. Eng. Div. ASCE

Assume:
(a) Type 5 Land Use, 36% impervious
(b) 1/16" depression storage on 36% area
(c) 1/4" depression storage on 64% area

0.25 inch volume of depression storage on such areas. The runoff from the pervious area is added to the runoff from the impervious areas to obtain the total runoff.

The cumulative total runoff must supply the volume of water which is transported out of the watershed by the sewers, plus the volume which accumulates as "neighborhood storage." This calculated cumulative "neighborhood storage" is shown in the last column of Table A-3.

The analytical procedures which have been described were used to evaluate the performances of present and proposed drainage systems, using both the hypothetical storms developed from Exhibit A-1 and certain historical storms described in the following section.

Computed Runoff From Historical Storms

Rainfalls of One Inch or More

All rainfalls exceeding one inch which occurred during the period 1871 - 1965 were analyzed to determine the expected frequency and extent of use of the underground storage system of the Deep Tunnel Plan. Exhibits A-3 to A-7 show the computed volume of runoff for the period, expressed as inches over the watershed. The runoff was calculated from historical recorded rainfall on the basis of simplified relationships developed by the City of Chicago Bureau of Engineering. Exhibit A-8 shows such a curve for Type 5 land use, which is defined as having 36 percent impervious area. For example, this relationship gives 3 inches of runoff for a 4.5 inch rainfall. This may be compared to the measured runoff of 3.1 inches from the July, 1957 storm which dropped about 6.5 inches of rainfall on the Chicago area. The curve of Exhibit A-8 thus indicates the maximum amount of runoff to be expected for a given storm for the watershed conditions assumed.

A total of 634 storms exceeding 1.0 inches of rainfall were identified in the 1871 to date records, a period of 94 years. Of these, only 30 storms produced a runoff of one inch or more. Only seven storms produced a runoff of two inches or more. The volume of the underground reservoir in the recommended plan is 2.5 inches.

Major Storms

Five historical storms were examined in detail. These included two storms in October 1954, one in July 1957, and two in September 1961. These storms are shown on Exhibits A-9 through A-13, in the form of cumulative hyetographs. Also shown on each of these exhibits are lines indicating the net rate of runoff into the sewer system, with the difference between the lines representing infiltration and depression storage. The exhibits also show lines indicating the rate of removal of storm water from the watershed. The light solid rate-of-removal lines represent the capacity of the present system of sewers (0.25 inches per hour), while the light dashed lines represent the capacity of the sewers and tunnels which comprise the proposed Deep Tunnel system. The vertical distance between the line representing runoff and the rate-of-removal lines represents the volume of water which must be stored temporarily somewhere on the wateshed. A comparison of resulting storage volumes under present and proposed system capacities is as follows:

Storm	Neighborhood Storage Volume with Present System (inches)	Neighborhood Storage Volume with Proposed Deep Tunnel System (inches)
October 12-13, 1954	1.4	0
July 12-13, 1957	1.5	0.1
September 1961	0.75	0

Typical Annual Array of Storm Runoffs

In addition to the rainfalls of one inch or more, there are many other smaller rains which produce small amounts runoff. A typical year is shown in Table A-4.

The array of storms listed in Table A-4 provides a means of estimating total average annual runoff. Of this total, only part will find its way into the tunnel system. For the sake of conservatism, however, it was assumed that all storm runoff would be handled by the tunnels. This assumption results in a total average annual runoff into the tunnels of 12 inches.

The average spacing of rains of 0.01 inch or more may be estimated on the basis of an average of 32 inches of rainfall per year. The wettest month, June, accounts for about 1/8 of the total. Assuming 1/8 of the 120 average annual rainfall events also occur in June, about half of the days in June would have some rain. This gives a perspective on the frequency with which storm runoff would enter the tunnel system during the wettest month.

Hydraulic Characteristics of Existing System

The Calumet Area which has been studied for the Deep Tunnel Plan is a 34-square-mile area in the City of Chicago bounded, roughly, by 87th Street on the north, by Lake Michigan and the Indiana line on the east, by 138th Street on the south and by Halsted Street on the west. Parts of the area extend somewhat beyond these boundaries, as is shown on Exhibit A-14. Within this area is the 21-square-mile drainage area of the initial stage proposed in Plan 1C.

The existing land use in the area is shown in Table A-1. The area is presently served by the type of combined sewer system shown

Table A-4

TYPICAL ANNUAL ARRAY OF STORM RUNOFFS

Rainfall Depth (Inches)	Average Runoff Depth	Number of Events Per Year	Total Runoff Contributed (Inches)
0 to 0.1	0.017	57.5	0.95
0.1 to 0.2	0.05	20.5	1.02
0.2 to 0.4	0.10	19.8	1.98
0.4 to 0.6	0.17	9.6	1.60
0.6 to 0.8	0.24	5.2	1.30
0.8 to 1.0	0.32	3.2	1.02
1.0 to 1.5	0.44	4.3	2.00
1.5 to 2.0	0.63	1.5	0.94
2.0 to 3.0	0.83	0.8	0.66
3 to 4		0.07	0.10
4 to 5		0.05	0.10
over 5		0.02	0.10
Total		120	11.77 or, say, 12 inches per year

NOTES: 1) Frequency of storms of 1 inch and over was developed from actual records of 94 years in Chicago.

2) Frequency of storms of less than 1 inch was developed from an analysis of the years 1953 through 1962, a period of 10 years.

in the upper right of Exhibit A-15. The smallest sewer is usually 12 inches in diameter, with a typical lateral sewer in each street being comprised of 660 feet of 12-inch, followed by 660 feet of 15-inch. At the end of this typical 1320-ft lateral sewer is a main sewer, usually 24 inches and larger in diameter.

The main sewers originally carried the drainage from the laterals to the river. When collection and treatment of sewage was instituted about 40 years ago, intercepting sewers were constructed along the rivers to intercept the dry weather flow of the main sewers and to convey it to treatment plants. Storm flows continued to overflow into the rivers, carrying some raw sewage along.

Because of the low and flat topography of the area, some main sewers presently discharge into pumping stations (such as the one at 95th St.) which then elevate the water for discharge into either the intercepting sewer or the river. Dry weather flows are pumped by one set of pumps into the intercepting sewer which then conveys the sewage to the treatment plant. Peak storm flows are pumped to the river by means of separate storm water pumps.

The hydraulic capacities of the present sewer systems range from 0.06 to 0.26 of an inch of runoff water per hour. The more intense storms overload the present system and cause occasional backing up into basements which are directly connected to the system. One of the major purposes of the proposed Deep Tunnel Plan is to eliminate this type of occurrence.

Hydraulic Characteristics of Deep Tunnel Plan

The proposed Deep Tunnel Plan is comprised of a number of elements. These are described in the following paragraphs in the order in which water would flow, as shown on Exhibit A-15.

Auxiliary Relief Sewers

These sewers would be constructed to relieve present laterals at their upper ends and to relieve main sewers at strategic points. An overall discharge capacity of about 2 inches per hour (2 cfs/acre), would be provided. The auxiliary relief sewers would function in much the same manner as envisioned by Messrs. Tholin and Keifer when the program of Auxiliary Relief Sewers was conceived many years ago. Every present sewer would be relieved.

Vertical Drains

The vertical drains would intercept the auxiliary relief sewers at strategic locations so that the relief sewer would be kept to an economical length, size and depth. By keeping the auxiliary relief sewers relatively short, their cost is substantially reduced. Furthermore, such sewers can be constructed below existing intersecting utilities, thus minimizing the need to relocate them, as the outlet end of the sewer can be made as low as desired.

Tunnels

These tunnels would receive water from the vertical drains and convey it to a reversible pumped-storage hydroelectric plant. Such tunnels would be constructed in the best formation available, the Galena-Platteville dolomite. The volume of the tunnels, combined with some bulk excavation, would be large enough to contain about 2.5 inches of runoff from the tributary area.

Pumping-Generating Station

This reversible plant would be used both for pumping storm water and for generating hydroelectric power as would a conventional pumped-storage peaking plant.

Surface Reservoir

This reservoir would be constructed on a flat area using dikes as high as required to contain the desired volume of water. This reservoir would serve several purposes:

(a) The storage of water for power generation

(b) The storage of water for flood control

(c) The storage of water for treatment

Treatment Facilities

The deep tunnel system itself would probably be designed to treat the waste water from the combined sewers prior to its being discharged into the waterways. Possible means to achieve the desired treatment are discussed in Appendix C.

The hydrological behavior of each of these elements of the present and proposed drainage system will be described in the paragraphs which follow.

Description of Hydrological Analyses

House drainage. Household wastes, roof drainage, and foundation drainage all are passed through the house sewer to the lateral sewer in the street. Usually, this house sewer is directly connected to the lateral with no provision for preventing reverse flow. Some houses have valves, standpipes, pumps, or other devices to prevent reverse flow. As more owners install these devices, increased burdens are placed on the remaining unprotected basements, as the same total volume of storm water must be accommodated somewhere. The only permanent and effective solutions involve first of all finding a location for storage of the unwanted water, and then providing a means of transporting it to that location at sufficiently high rates of flow.

The typical piezometric levels under existing conditions and with the proposed Deep Tunnel Plan are shown on Exhibit A-16. This demonstrates the basement flooding potential that exists. Under present conditions the volume of space in basements available for the temporary storage of storm water constitutes an important element of the hydrological system. It must be taken into account in any analysis which is to be realistic. This volume was estimated on the basis of field surveys made in the course of the studies reported here. Data from these surveys are summarized on profiles of the lateral sewers, shown on Exhibits A-17 through A-19.

The elevation of each basement was taken to be eight feet below the first floor elevation determined from the field survey. The average basement area was taken to be 840 square feet. The total volume of all basements connected to a given lateral was calculated as a function of the elevation of the water above the elevation of the invert of the lateral outlet. The results of these calculations are shown on Exhibit A-20. This information was used in calculating the performance of the lateral sewers under both present and proposed conditions.

Street Drainage. The street pavements are crowned with concrete curbs and gutters along both sides. The slopes of these gutters are typically 1 foot in 200 feet, with inlets being placed at low points. Normally, about 12 inlets are connected to a 1320-ft lateral. Each inlet drains an average area of about 6000 square feet of pavement and connected driveways, plus whatever other area is drained overland onto these pavements. The inlets are of the grated type with curb openings in many instances. Because of the flat slopes, water can pond over the inlets and, therefore, the inlets have a large effective capacity.

A typical gutter feeding an inlet will, during a severe storm, deliver water from about a half-acre area. Assuming runoff at 2 inches per hour, a discharge of 1 cubic foot per second would result. With a crown slope of 0.06, a gutter slope of 0.005 and a Manning "n" value of 0.015, such a flow would be associated with an average cross sectional area of about 0.3 square foot per gutter. Multiplied by a length of 3200 feet of gutter in a 10-acre watershed, this gives a volume of 0.03 inch spread out over the watershed.

Lawn and Yard Drainage. Overland flow over yards, walks, and alleys can store an appreciable volume of flow, particularly on flat,poorly graded areas. It is difficult to estimate this volume accurately. For purposes of this report, a value of 0.15 inch was used, which is believed to be conservatively small. This is in addition to the volume of water known as depression storage, which is trapped permanently in low spots and subsequently percolates into the soil. (See subsequent section entitled "Questionnaire Survey", where unusually large yard storage in one sample neighborhood is discussed.)

Lateral Sewers. The 15-inch diameter outlet of the present lateral discharges drainage from a 10-acre watershed. With a difference in elevation of 6 feet between the lateral outlet and the lowest basement, the average slope of the piezometric head line would be about 0.005. However, because of the variable discharge along the length of the sewer, the piezometric slope at the outlet is greater. This provides an outlet velocity of about 5 feet per second and an outlet discharge of 6.3 cubic feet per second. This is a unit discharge of 0.63 cfs/acre, or 0.63 inches per hour, which is smaller than required to handle even a 2-year storm. Furthermore, the 0.63 inch/hour rate is attained only if the main sewer is drawn down to receive the discharge from the lateral. In many severe storms the main sewer is also overtaxed so that the potential capacity of the lateral is not achieved.

Under the proposed future conditions associated with the Deep Tunnel Plan, the lateral would be provided with a free outlet at the upper end. The main sewer would be relieved sufficiently to provide a free outlet at its lower end also. The lateral would then discharge at both ends under steeper gradients, as shown on Exhibit A-16. It is estimated that the discharge rate at the 12-inch end would be about 5.25 cfs, that at the 15-inch end it would be about 10.5 cfs, and the total would be about 15.75 cfs, without basement flooding. Serving an area of 10 acres, the lateral thus provides a drainage capacity of 1.6 inches per hour, or more than double the potential capacity of the present lateral. Furthermore, as the main sewer is also relieved, the lateral is actually able to attain its full potential capacity.

Design relief capacity of main sewers would actually be 2.0 inches per hour instead of 1.6 inches per hour, to provide a little spare capacity which could prove useful under extreme conditions. Inasmuch as it is not difficult to achieve such high capacities with the proposed system it was judged worthwhile to do so.

Main Sewers. The present main sewers are capable of handling storm drainage at rates on the order of 0.06 to 0.26 inches per hour, with a typical average of about 0.17 inches per hour. This is much smaller than the present 0.63 inches per hour capacity of the laterals. If the volume of runoff exceeds the volume of the main sewer, the maximum discharge capacity of the system is reduced to about 0.25 inch per hour, and less in many instances.

With the Deep Tunnel Plan, the present main sewers would be relieved at strategic points to increase the capacity to about 2 inches per hour. Segments of existing main sewers would thus serve smaller drainage areas. Present main sewer slopes are flat enough in most instances so that substantial lengths would, during storm periods, flow in an opposite direction to the direction of normal flow. Relief of main sewers would be achieved by new interceptors and vertical drains.

Vertical Drains. Inlets to vertical drains would be designed to minimize the space required in the street. Inasmuch as some of these vertical drains will have discharge capacities in excess of 500 cubic feet per second, careful hydraulic design is required to minimize the space required and to provide for efficient passage of water and debris. Inlets may be designed to operate as siphon spillways with a minimum of air entrainment, or they may be designed to pass debris without regard to the volume of air entrained. Exhibit A-21 illustrates the types of design to be studied.

The flow in the barrel of the vertical shaft would be a mixture of water, air, and water vapor requiring perhaps 15 seconds to fall to the bottom of the shaft where some decelleration would take place (back to atmospheric pressure) just before the flow would discharge from the lower end of the shaft. The barrel would be lined with steel.

The high-velocity vertical jet of water, air, and debris would impinge into a stilling basin excavated in the rock adjacent to the tunnel as shown in Exhibit A-22. Hydraulic model studies of this feature would be required in final design.

Tunnels. The maximum discharge capacity of the tunnels is limited by the permissible design velocity. This is true because of the large head available for producing flow through the tunnels. Assuming a velocity of 25 feet per second is the maximum desirable velocity and that a gradient of 1 percent is available, the following discharge capacities are calculated;

Tunnel Dia.	Blasted Tunnels, n=0.033		Smooth bore tunnels, n=0.015	
10'	V=20.5;	Q=1,620	V=25;	Q=1,960
20'	V=25;	Q=7,850	V=25;	Q=7,850
30'	V=25;	Q=17,500	V=25;	Q=17,500
40'	V=25;	Q=31,400	V=25;	Q=31,400

Pumping-Generating Station. The initial power installation in the recommended plan would be 300 MW. The corresponding pumping capacity would be about 3750 cubic feet per second.

The future expansion of the hydroelectric plant to perhaps four times its initial size would require a much enlarged tunnel system. Presumably, this would be developed in the course of extending the system to the remainder of the metropolitan area. Likewise, additional tunnels and turbines would provide for the required increased discharge capacity.

The 300 MW pumping plant operating at 3000 cfs during off peak periods would evacuate 2.5 inches of runoff from the tunnels in a period of 10 hours. Thus, it would be possible to empty the tunnels completely during typical off-peak periods which are commonly at least 18 hours long.

Surface Reservoir. The volume of the surface reservoir could be determined on the basis of provisions for future expansion. A volume of 2360 acre-feet was selected for the recommended plan. Ultimately, a volume of 10,000 acre-feet might be required for a fully enlarged system.

Evaporation and leakage from the upper reservoir would be at most on the order of 2 to 3 acre-feet per day or, roughly, 1 mgd. The leakage into the tunnel system is estimated to be greater than this (see Appendix D), indicating there could be no problem of make-up water even during a drought period.

The ungated spillway and outlet from the reservoir would be sized to spill water at a rate of flow equal to the total pumping capacity of the pumping station so that the dikes could not be overtopped through an operational error. Thus the spillway capacity in the recommended plan would be 3750 cfs.

Waterways and Lake Michigan. With the conventional sewer system, flood flows in the waterways are increased as the system is expanded. This brings with it an ever increasing threat of polluting Lake Michigan. By contrast, as the Deep Tunnel Plan would be expanded, flood flows in the waterways would be decreased and the threat of polluting Lake Michigan would be eliminated.

Hydrologic Evaluation of Deep Tunnel Plan

Exhibit A-23 presents a family of curves summarizing storage requirements as related to storm recurrence interval. Assumed rates of outflow of storm water from the neighborhood are labeled in inches per hour of outflow capacity of the sewer system. For example, a capacity of one-inch per hour is sufficient to remove in one hour a volume equal to a sheet of water one-inch in depth spread over the entire area of the watershed. On the left of this exhibit is the scale indicating volume of storage required. Across the bottom is a scale indicating recurrence interval of the hypothetical storm used in the calculation. Also drawn on the exhibit is a line labeled "volume of lateral, gutter, and overland flow." This line indicates the estimated volume of water contained in the lateral, in the flowing gutters, and in the sheet of water which moves across the surface of the ground on its way to the inlets in the street.

Exhibit A-23 may be used in the following manner. For example, assume it is desired to determine the volume of temporary neighborhood storage that would be required to handle a storm with a recurrence interval of 10 years for a sewer system capacity of 0.25 inches per hour.

Entering the chart at the bottom at a 10-year recurrence interval, and running vertically along this line, 0.18 inches is read as being the volume of water in the lateral, the gutters, and in overland flow; and running further up this vertical line to the intersection with the line labeled "0.25 inches per hour" a value of 1.76 inches is read as being the total volume of temporary storage required. The difference between these two figures is 1.58 inches. This is the excess volume of water which must be located somewhere on the watershed for the conditions assumed.

Exhibit A-23 was used to evaluate the effect of the Deep Tunnel Plan on the magnitude of the volume of water which would be stored temporarily in the watershed. Table A-5 summarizes the results of the analytical studies made to date. Table A-5 indicates only minor requirements for neighborhood storage for storms equal for or greater than a 25-year recurrence interval. Discharges of flood water into the waterways would be reduced to levels which would not cause flood damage or transportation interruptions.

In considering the neighborhood storage requirements of Table A-5, two points should be borne in mind. First, the theoretical extent of storage requirements are based on conservative assumptions throughout the analytical procedure, i. e., the indicated storage requirements are apt to be higher than actually will result, secondly, the neighborhood storage requirements can, at least in part, be met by some of the components of neighborhood storage other than basement flooding. Thus it should not be assumed that some basements will have to have all of the neighborhood storage. It is quite likely that the combination of these two considerations will mean that basement flooding will be virtually eliminated for all storms of less than 100-year frequency.

Table A-5

EFFECT OF THE DEEP TUNNEL PLAN ON THE VOLUME OF WATER REQUIRED TO BE STORED TEMPORARILY IN THE URBAN WATERSHED OF THE CALUMET AREA

Recurrence Interval of Storm Years	Occurrences per 100 Years	Neighborhood storage req'd with present system (inches)	Neighborhood Storage required with proposed system (inches)	Percent reduction in Neighborhood storage required	Difference in neighborhood storage required (inches)	Difference in neighborhood storage times occurrences (inches)
100	1	1.65	0.49	70%	1.16	1.16
50	1	1.22	0.23	80%	0.99	0.99
25	2	0.98	0.15	83%	0.83	1.66
10	6	0.70	0	100%	0.70	4.20
5	10	0.45	0	100%	0.45	4.50
2	30	0.14	0	100%	0.14	4.20
1	50	0.04	0	100%	0.04	2.00
					TOTAL	18.71

Questionnaire Survey

A survey was made by mailing a questionnaire to the majority of approximately 1000 households in the three sample neighborhoods studied. One hundred sixty replies were scrutinized to see if the analytical procedure used in these studies appeared to be valid. The replies to the questionnaires were sorted by neighborhood and compared with the predictions of the analytical model. A specific comparison was made of the calculated depths of basement flooding under existing conditions (house by house) for the major historical storms (1954 and 1957) with the reported depths.

The results are tabulated in Table A-6. In two of the three sample neighborhoods the predicted and reported depths agreed well. In the third, there was no agreement. This third neighborhood, which showed less basement flooding than would be calculated by the analytical model, was found to have two distinctive features not contemplated in the model:

(a) The houses drained to an alley sewer whereas the streets drained to a separate sewer in the street;

(b) the yards around the houses were commonly lower than the street pavements and provided a large volume of temporary storage space.

Thus, in the last instance neighborhood storage is believed to have been satisfied under existing conditions without use of basement storage.

A more extensive study of the local sewer system is required. to determine the reliability of predecting the component of neighborhood

Table A-6

RESULTS OF QUESTIONNAIRE SURVEY

Residence Code Number	Length of Residence (Years)	Predicted Depth (Inches)	Reported Depth (Inches)	Prediction Error Plus (Inches)	Prediction Error Minus (Inches)
1	13	18	18	--	--
2	13	9	4	+ 5	
3	9	10	6	+ 4	
4	14	9	48		-39
5	12	6	0	+ 6	
6	13	17	8	+ 9	
7	12	15	6	+ 9	
8	14	4	some	--	--
9	13	9	2	+ 7	
10	12	10	12		-2
11	8	0	1		-1
12	9	10	24		-14
13	26	9	14		-5
14	13	5	18		-13
15	18	7	18		-11
16	11	14	24		-10
17	13	10	6	+ 4	
18	12	19	18	+ 1	
19	20	10	6	+ 4	
20	9	9	9	--	--
21	12	7	12		-3
22	42	10	3	+ 7	
23	12	7	5	+ 2	
24	14	21	24		-3
25	14	13	4	+ 9	
		Totals		+67	-101

$$\text{Average error} = \frac{-34}{25 \text{ basements}} = -1.36 \text{ inches per basement}$$

Table A-6

RESULTS OF QUESTIONNAIRE SURVEY (cont'd.)

Residence Code Number	Length of Residence (Years)	Predicted Depth (Inches)	Reported Depth (Inches)	Prediction Error Plus (Inches)	Prediction Error Minus (Inches)
26	18	7	30		-23
27	11	3	2	+ 1	
28	15	13	7	+ 6	
29	12	13	3	+10	
30	28	2	0	+ 2	
31	21	23	18	+ 5	
32	13	15	12	+ 3	
33	15	8	0	+ 8	
34	10	0	2		-2
35	18	19	36		-17
36	27	11	0	+11	
37	19	0	6		-6
38	17	5	4	+ 1	
39	19	0	3		-3
40	20	0	0	--	--
41	19	3	12		-9
42	19	7	10		-3
43	30	3	0	+ 3	
44	13	6	0	+ 6	
45	24	6	0	+ 6	
46	51	0	3		-3
47	15	12	12	--	--
48	30	12	0	+12	
49	10	22	2	+20	
50	46	18	6	+12	
51	10	11	0	+11	
52	14	7	3	+ 4	
53	26	4	24		-20
54	12	13	0	+13	
55	13	7	6	+ 1	

Table A-6

RESULTS OF QUESTIONNAIRE SURVEY (cont'd.)

Residence Code Number	Length of Residence (Years)	Predicted Depth (Inches)	Reported Depth (Inches)	Prediction Error	
				Plus (Inches)	Minus (Inches)
56	10	0	2		-2
57	27	2	15		-13
58	12	8	10		-2
59	21	17	0	+17	
60	27	0	12		-12
61	15	7	36		-29
62	22	20	8	+12	
63	13	7	8		-1
		Totals		+164	-145

Average error = $\frac{+19}{38 \text{ Basements}}$ = +0.50 per basement

64	15	3	0	+3	
65	50	0	12		-12
66	42	22	0	+22	
67	20	0	0	--	--
68	60	0	0	--	--
69	25	10	6	+4	
70	30	0	0	--	--
71	18	0	0	--	--
72	60	16	0	+16	
73	32	18	0	+18	
74	52	71	0	+71	
75	10	7	0	+7	
76	40	7	0	+7	
77	25	0	0	--	--
		Totals		+148	-12

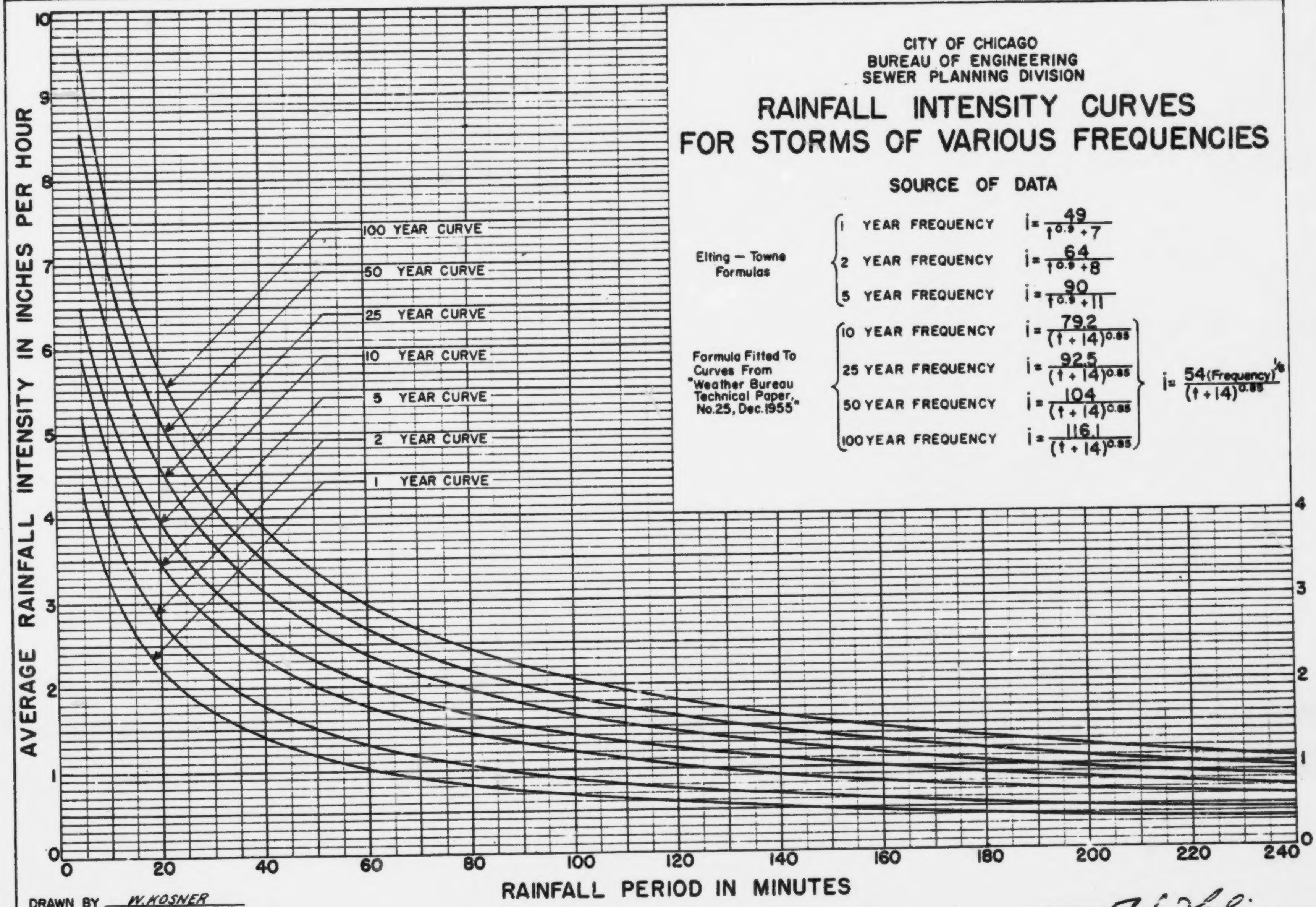
Average error = $\frac{+136}{14 \text{ basements}}$ = +9.73 per basement storage that can be satisfied without flooding of basements. This will be necessary before the analytical procedure can be used to predict any basement flooding that might remain with the Deep Tunnel Plan. The size of the sample areas should be increased, and more detailed information obtained concerning each house if more precise predictions are to be obtained. On the basis of the studies made to date, it is believed that the analytical procedure used is conservative in that it predicts maximum and not minimum amounts of neighborhood flooding.

Future Investigations

Exhibits A-21 and A-22 show two elements of the proposed plan which require hydraulic analyses to evaluate their performances. Flow in the deep vertical shafts is complex, involving water, air, water vapor, and debris. Undoubtedly new knowledge will result from research on this feature, probably by means of large-scale laboratory tests. Similar needs must be met for the energy dissipation basin, although this is less difficult to analyze once the flow in the vertical drains is understood. Neither of these two laboratory and analytical studies is considered to present any insurmountable obstacles.

More detailed information concerning the local drainage system is required to refine further the analytical procedures of the present and proposed systems. Once this is obtained, it will be possible to obtain a more detailed and faithful simulation of actual drainage experience. This, in turn, will provide a needed tool for designing the most appropriate solution to the actual drainage problem.

EXHIBIT A-1



CITY OF CHICAGO
BUREAU OF ENGINEERING
SEWER PLANNING DIVISION

RAINFALL INTENSITY CURVES FOR STORMS OF VARIOUS FREQUENCIES

SOURCE OF DATA

Elting — Towne Formulas

1 YEAR FREQUENCY $i = \frac{49}{t^{0.9} + 7}$

2 YEAR FREQUENCY $i = \frac{64}{t^{0.9} + 8}$

5 YEAR FREQUENCY $i = \frac{90}{t^{0.9} + 11}$

Formula Fitted To Curves From "Weather Bureau Technical Paper, No.25, Dec.1955"

10 YEAR FREQUENCY $i = \frac{79.2}{(t + 14)^{0.85}}$

25 YEAR FREQUENCY $i = \frac{92.5}{(t + 14)^{0.85}}$

50 YEAR FREQUENCY $i = \frac{104}{(t + 14)^{0.85}}$

100 YEAR FREQUENCY $i = \frac{116.1}{(t + 14)^{0.85}}$

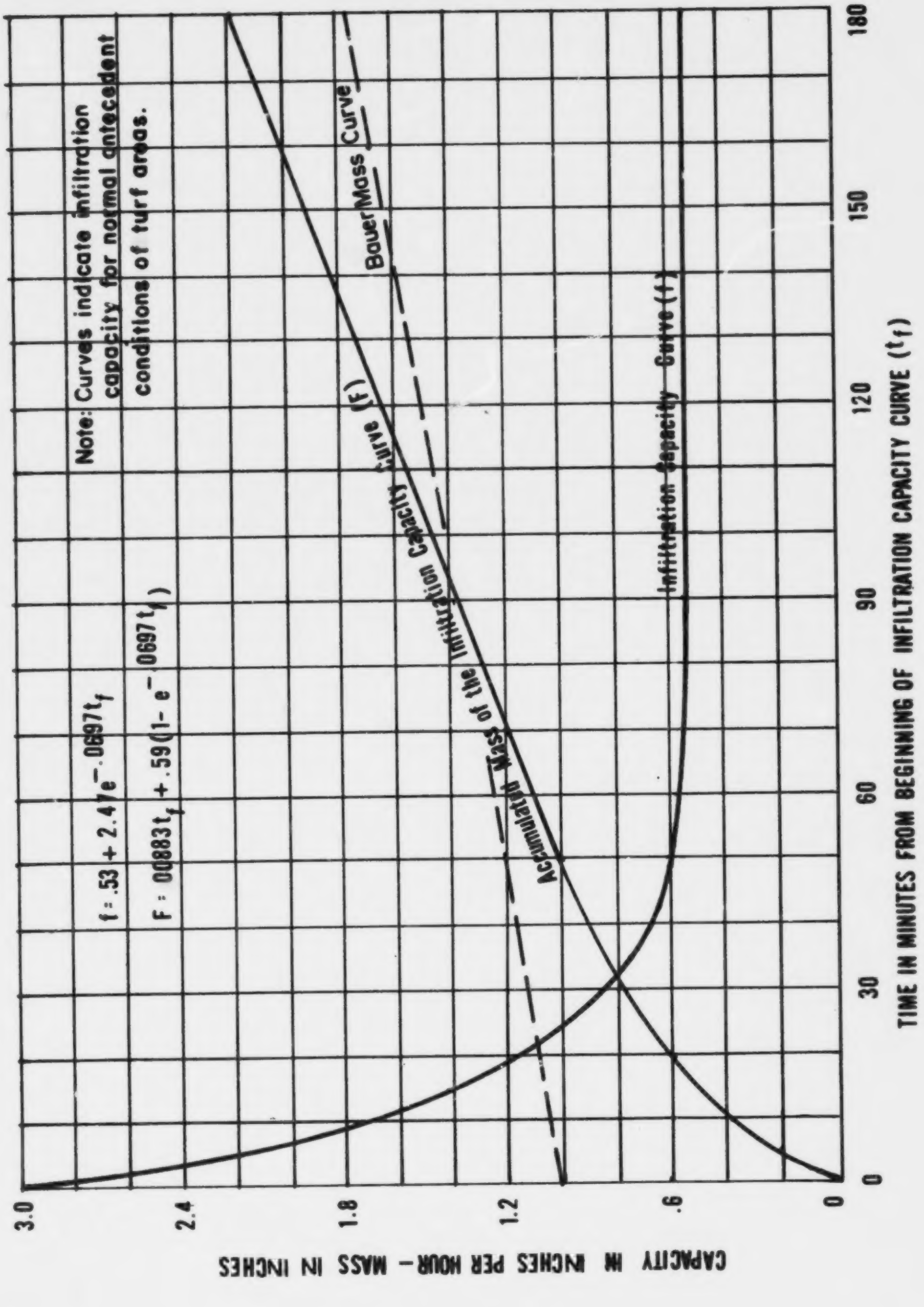
$i = \frac{54(\text{Frequency})^{1/6}}{(t + 14)^{0.85}}$

DRAWN BY W. KOSNER
COMPUTED BY G.G. TANAKA
EXAMINED BY Clint J. Keifer
Civil Engineer V

REVISED 10-57

APPROVED
Chief Sewer Engineer

DC-56



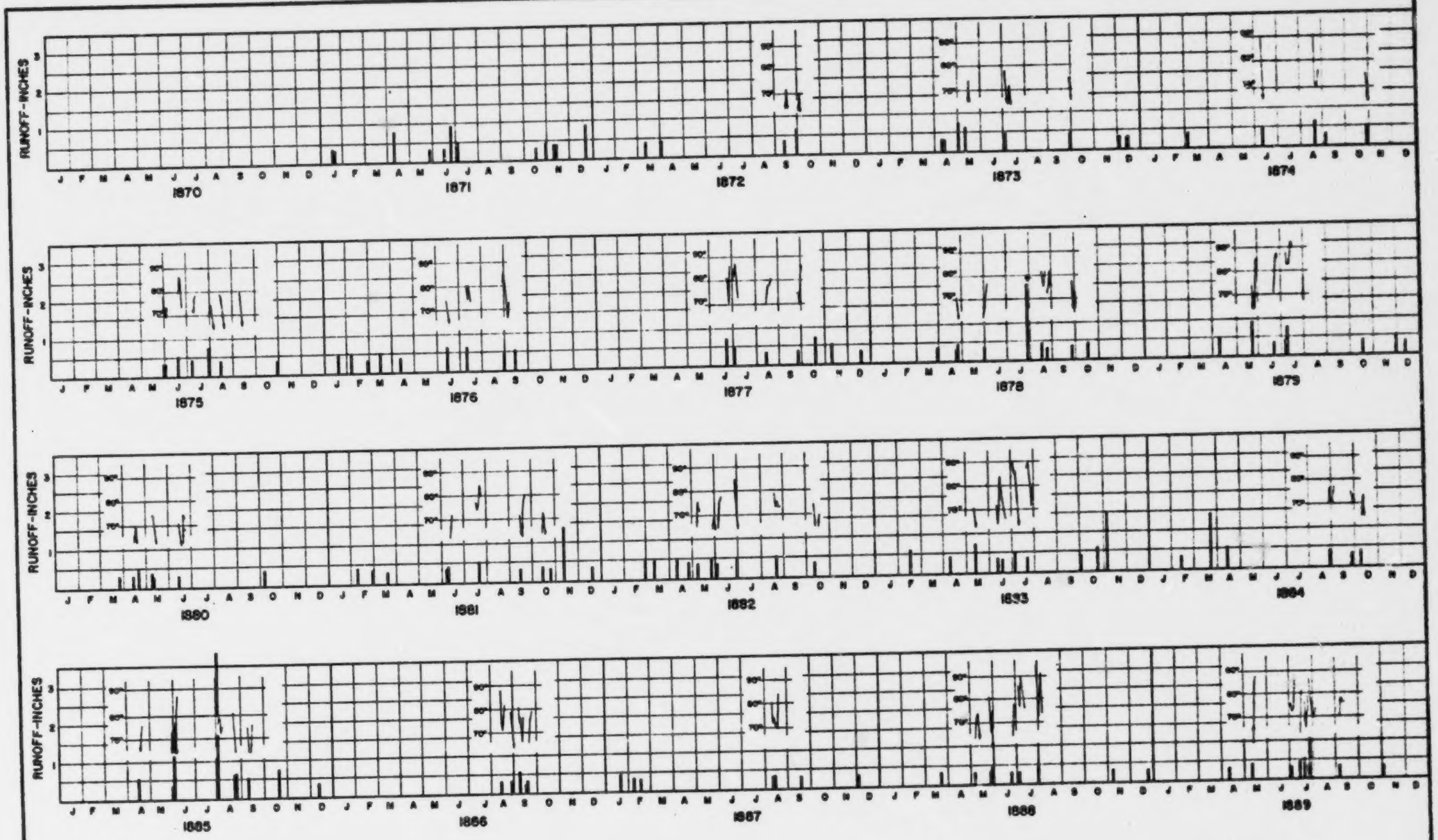
From A.L. Tholin, C.J. Keifer, Journal of Sanitary Engr. Division, A.S.C.E. Proceedings, Vol. 85, 1884. March 1959

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

INFILTRATION CURVES

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-3



CHICAGOLAND AREA

FLOOD & POLLUTION CONTROL STUDIES

COMPUTED HISTORICAL RUNOFF
SHEET I

LEGEND

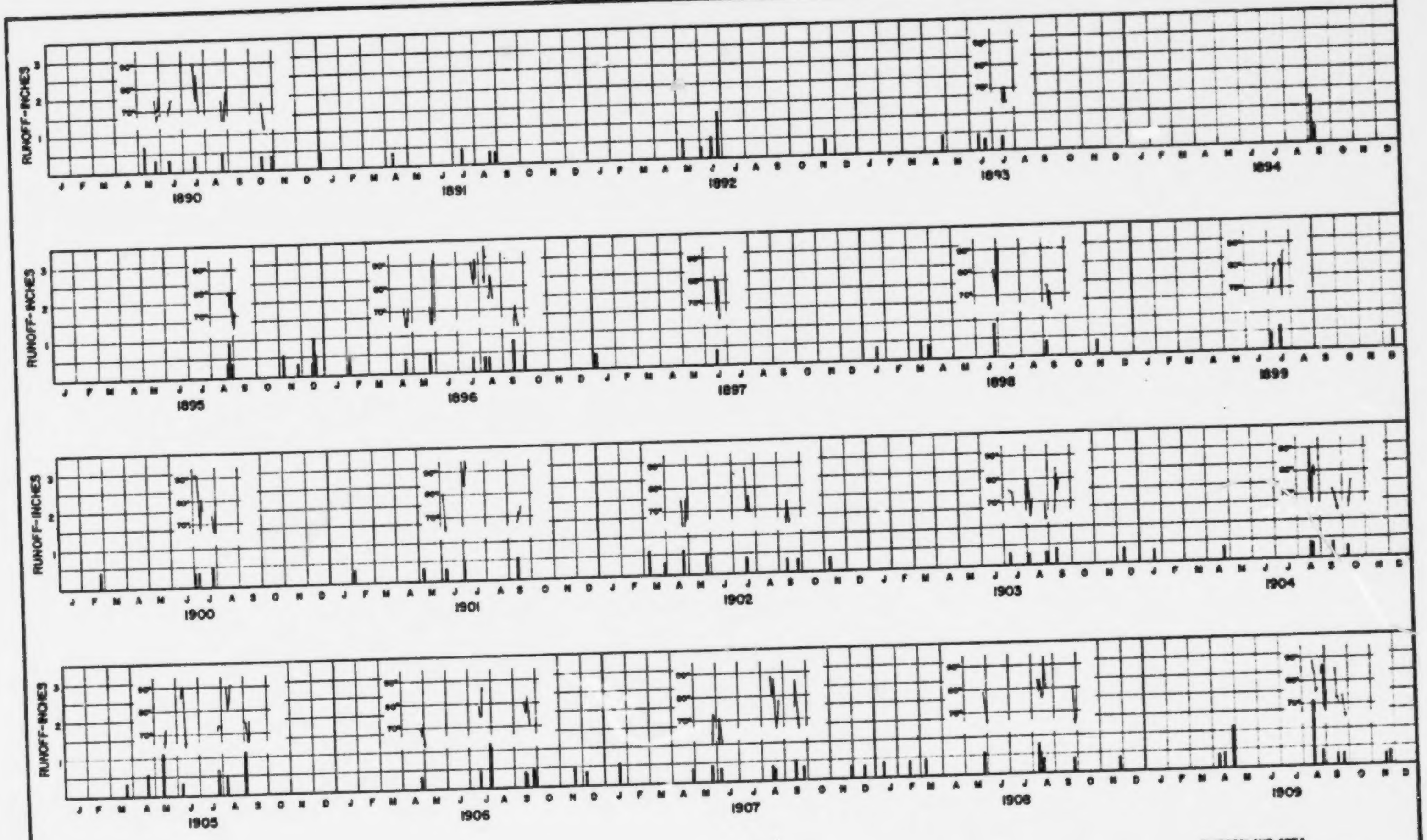
⊥ COMPUTED RUNOFF FROM HISTORICAL STORMS

| TEMPERATURE, SHOWN ONLY FOR DAY OF STORM IF GREATER THAN 70° F

HARZA ENGINEERING COMPANY DWG. NO. 387 P

BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-4



LEGEND

⊥ COMPUTED RUNOFF FROM HISTORICAL STORMS

/ TEMPERATURE, SHOWN ONLY FOR DAY OF STORM IF GREATER THAN 70° F

CHICAGOLAND AREA

FLOOD & POLLUTION CONTROL STUDIES

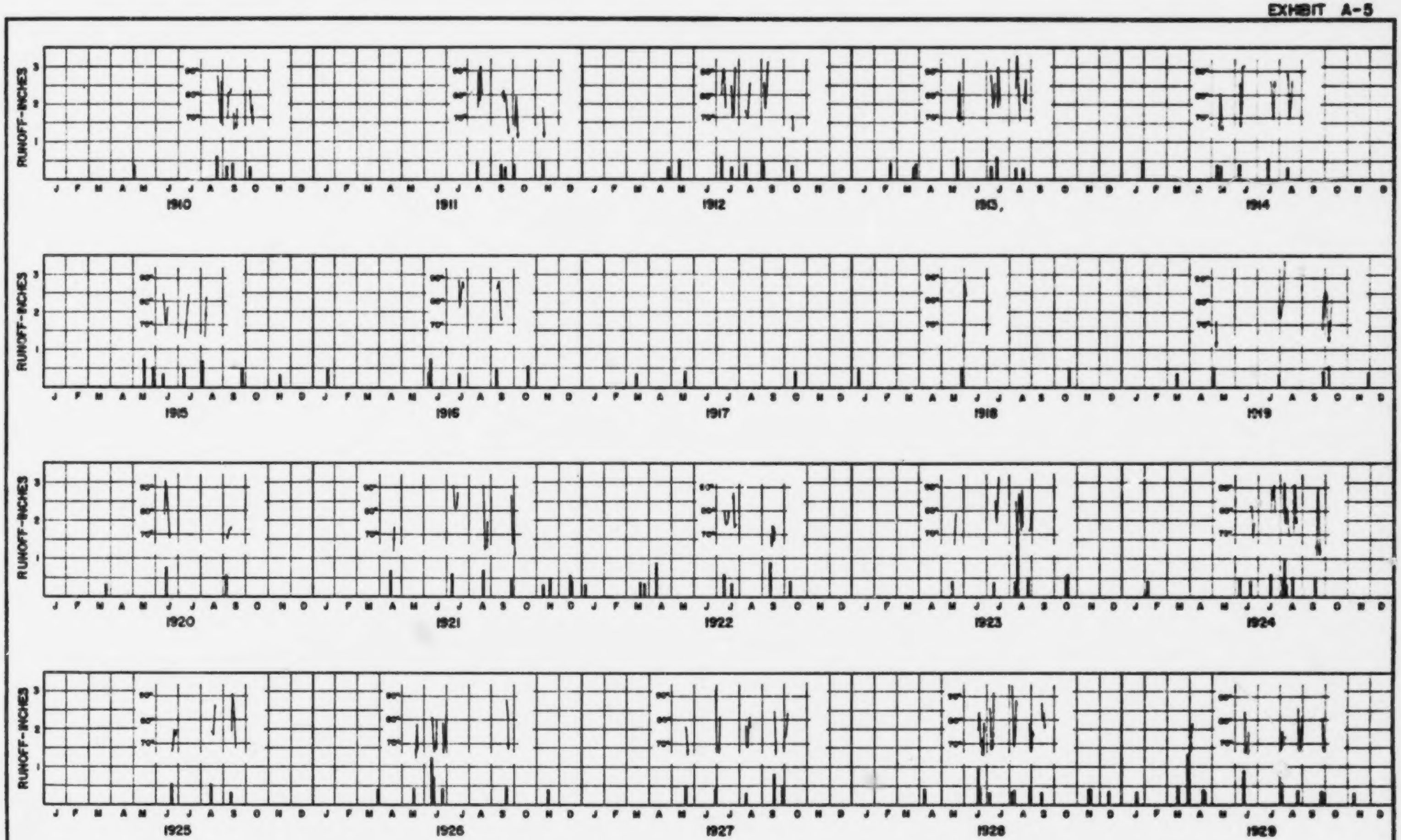
COMPUTED HISTORICAL RUNOFF
SHEET 2

HARZA ENGINEERING COMPANY DWG NO. 387 P

BAUER ENGINEERING, INC. PROJECT 6524

F 147

EXHIBIT A-5



CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

COMPUTED HISTORICAL RUNOFF
SHEET 3

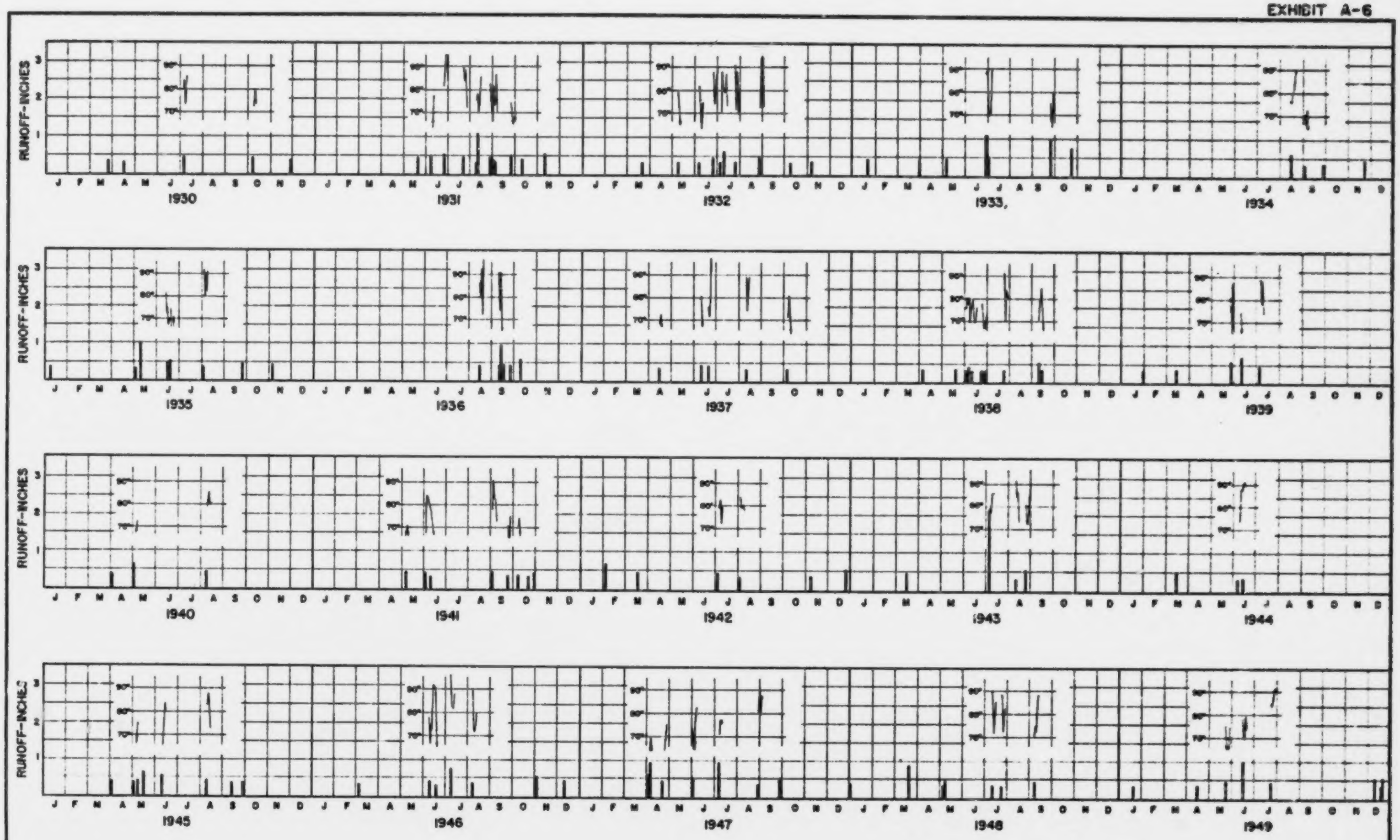
LEGEND

⊥ COMPUTED RUNOFF FROM HISTORICAL STORMS

| TEMPERATURE, SHOWN ONLY FOR DAY OF STORM IF GREATER THAN 70° F

HARZA ENGINEERING COMPANY DWG.NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6324

EXHIBIT A-6



LEGEND

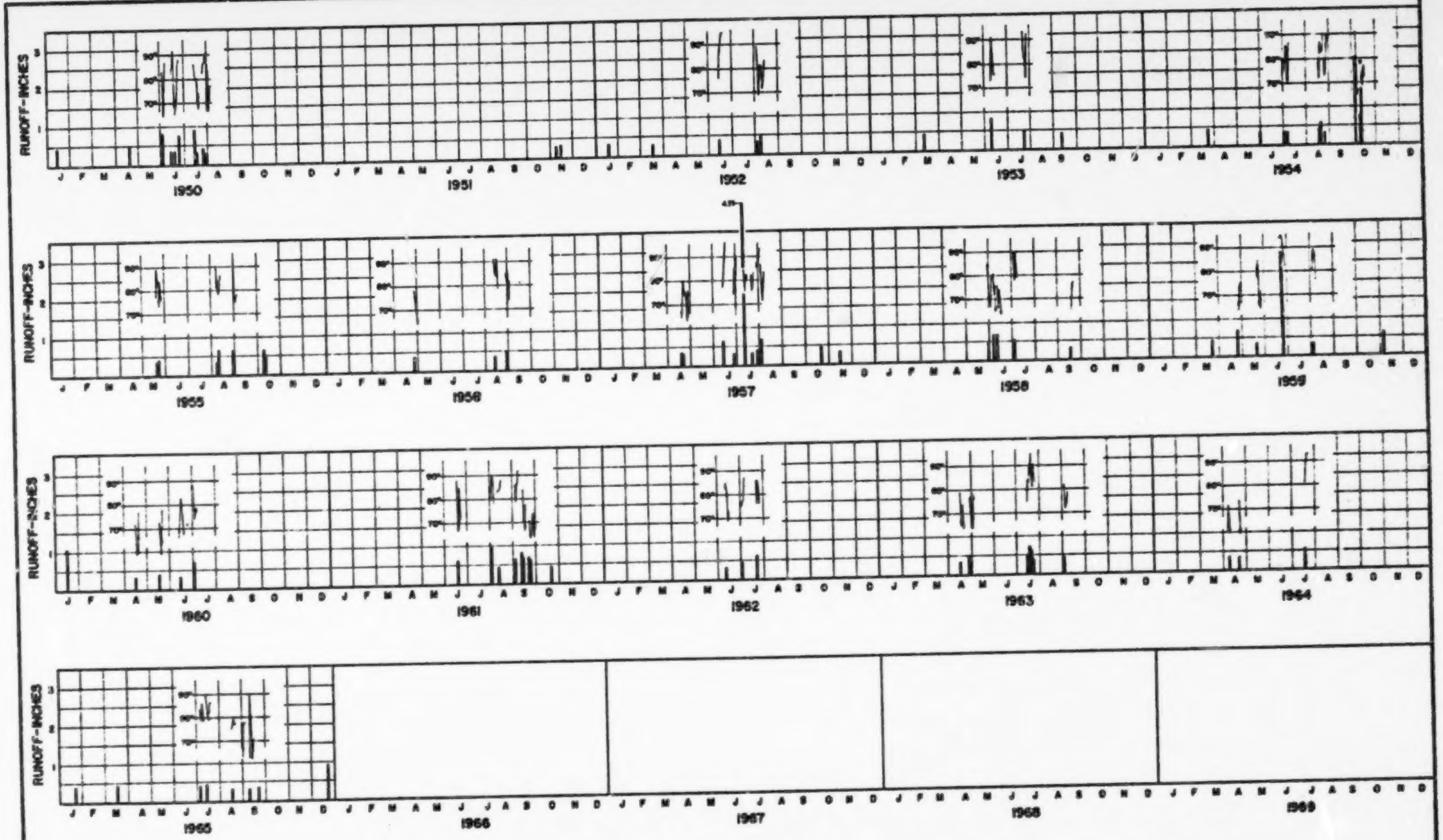
- COMPUTED RUNOFF FROM HISTORICAL STORMS
- TEMPERATURE, SHOWN ONLY FOR DAY OF STORM IF GREATER THAN 70° F

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

COMPUTED HISTORICAL RUNOFF
SHEET 4

HARZA ENGINEERING COMPANY DWG NO. 387 F
BAUER ENGINEERING, INC. PROJECT 6324

EXHIBIT A-7



LEGEND

⊥ COMPUTED RUNOFF FROM HISTORICAL STORMS

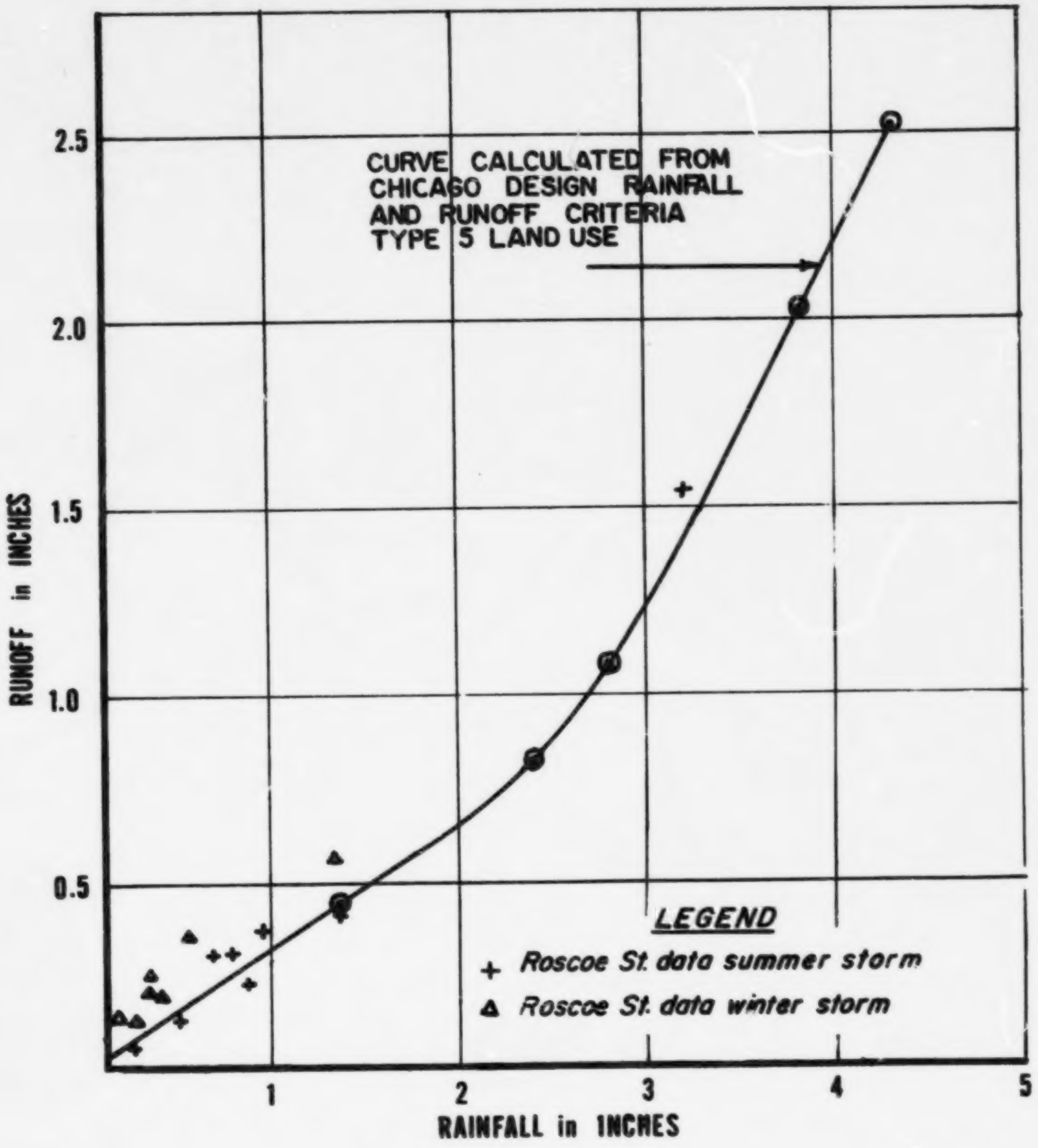
| TEMPERATURE, SHOWN ONLY FOR DAY OF STORM IF GREATER THAN 70° F

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

COMPUTED HISTORICAL RUNOFF
SHEET 5

HARZA ENGINEERING COMPANY DWG.NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-8

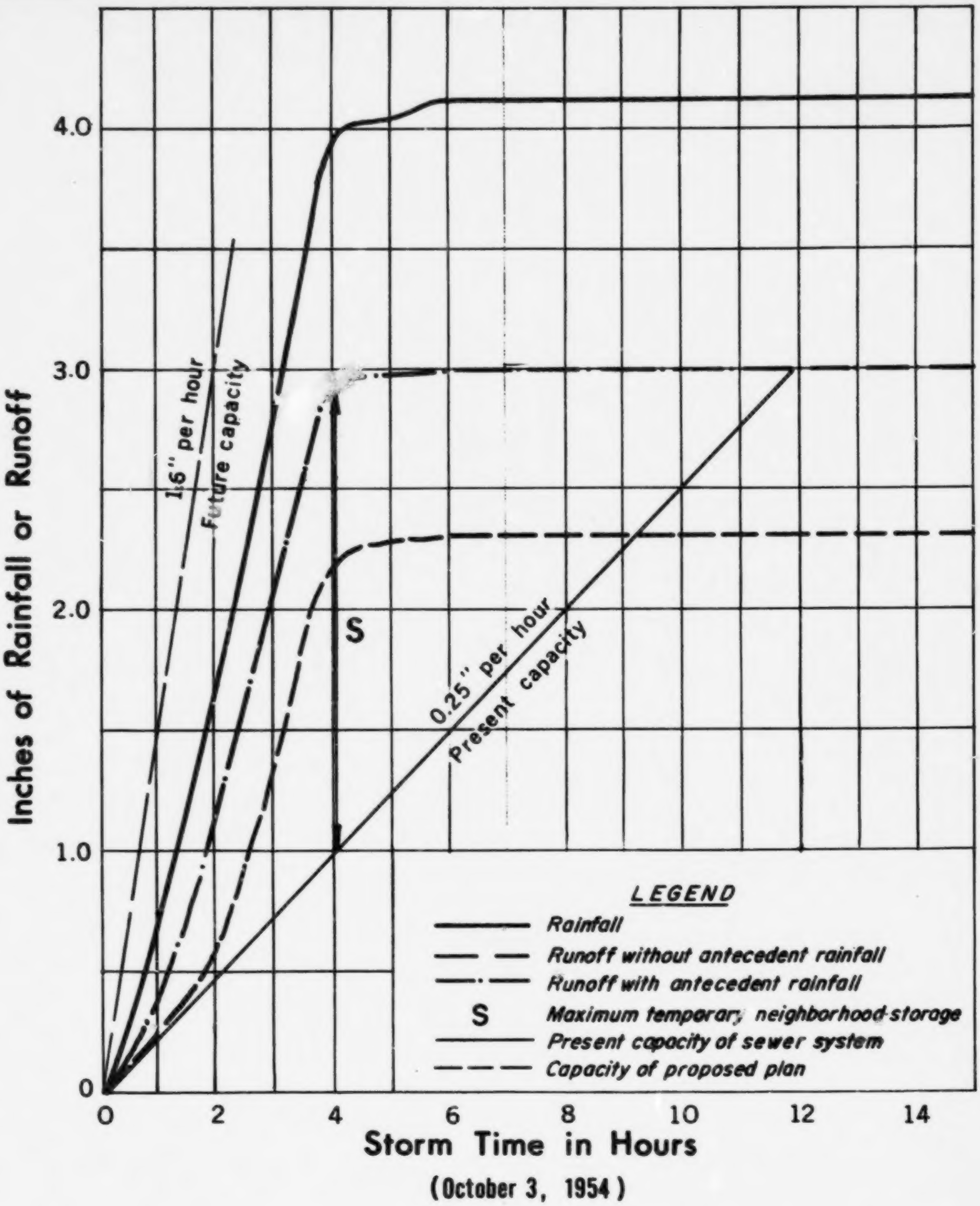


CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

RAINFALL-RUNOFF RELATIONSHIP

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A- 9

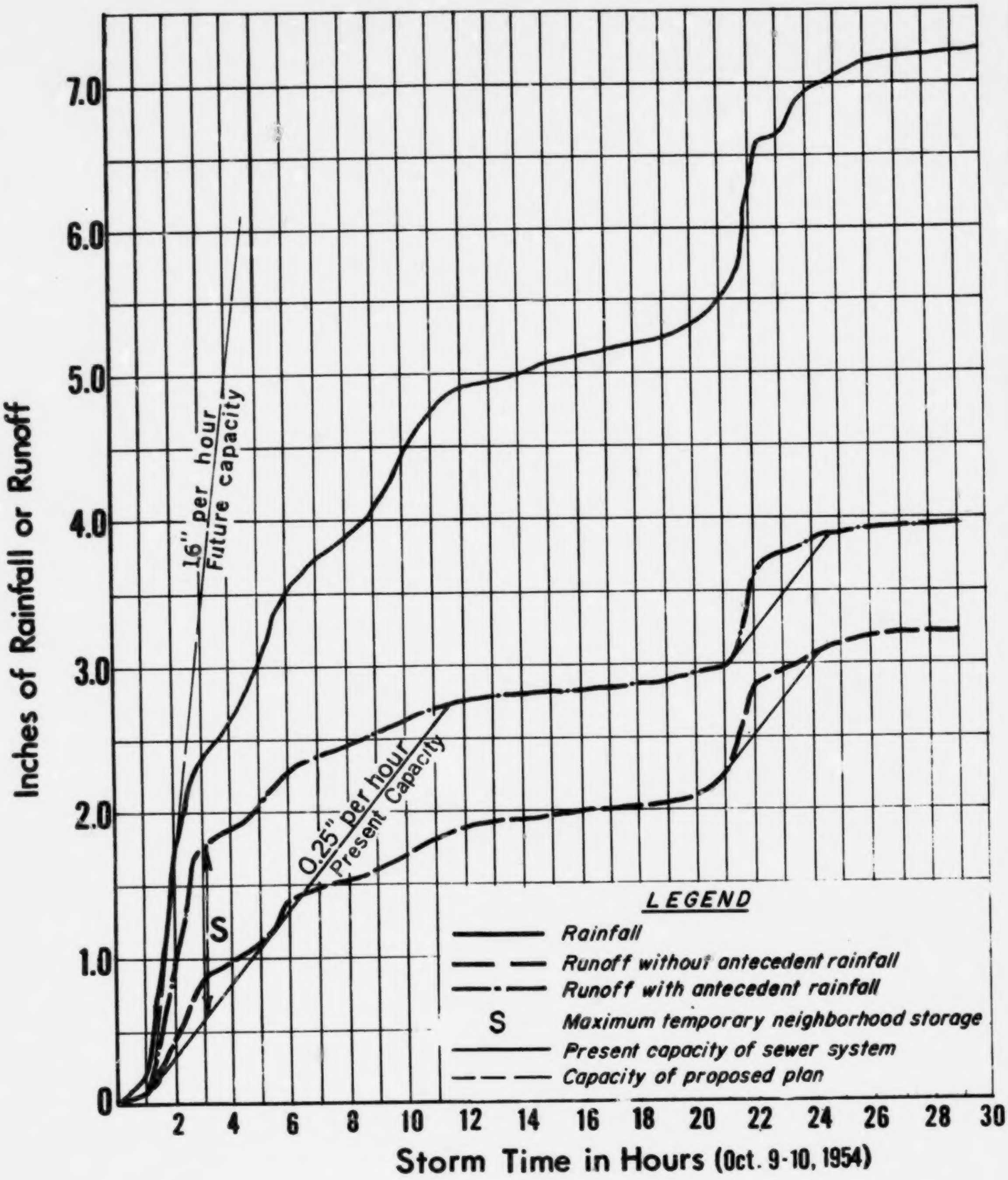


CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

HYETOGRAPH
Oct. 3, 1954

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-10

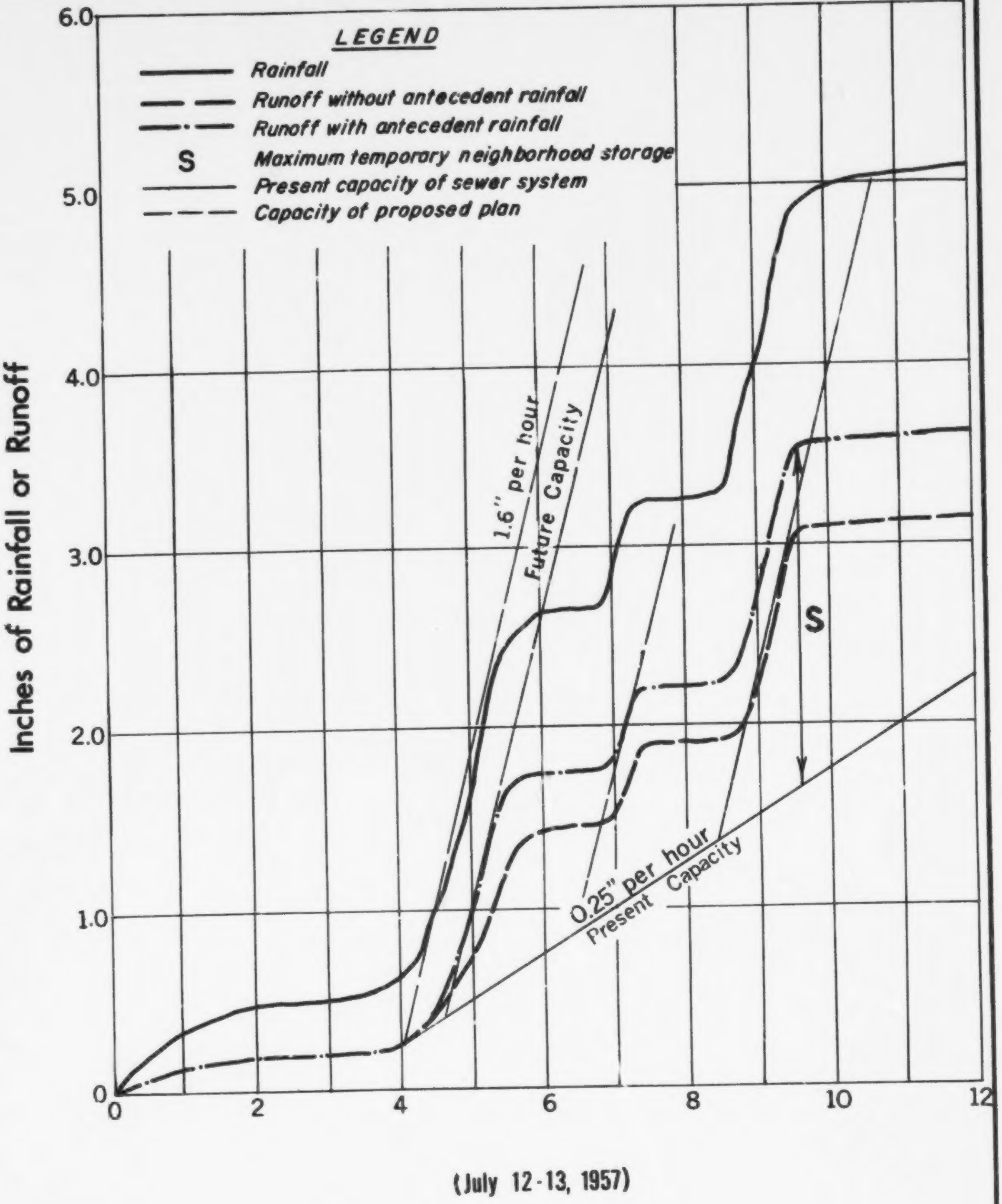


CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

HYETOGRAPH
Oct. 9-10, 1954

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-11

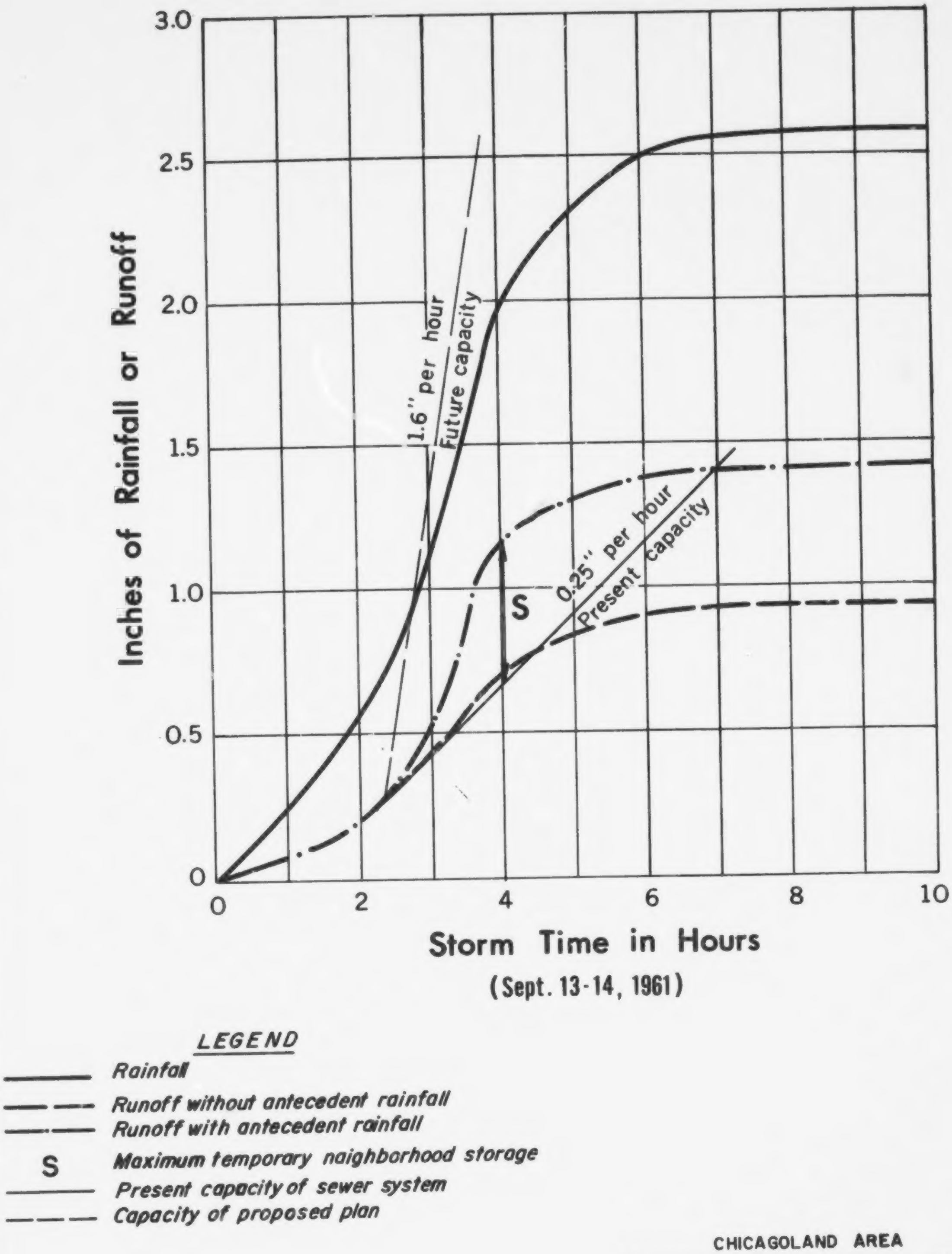


CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

HYETOGRAPH
July 12-13, 1957

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-12

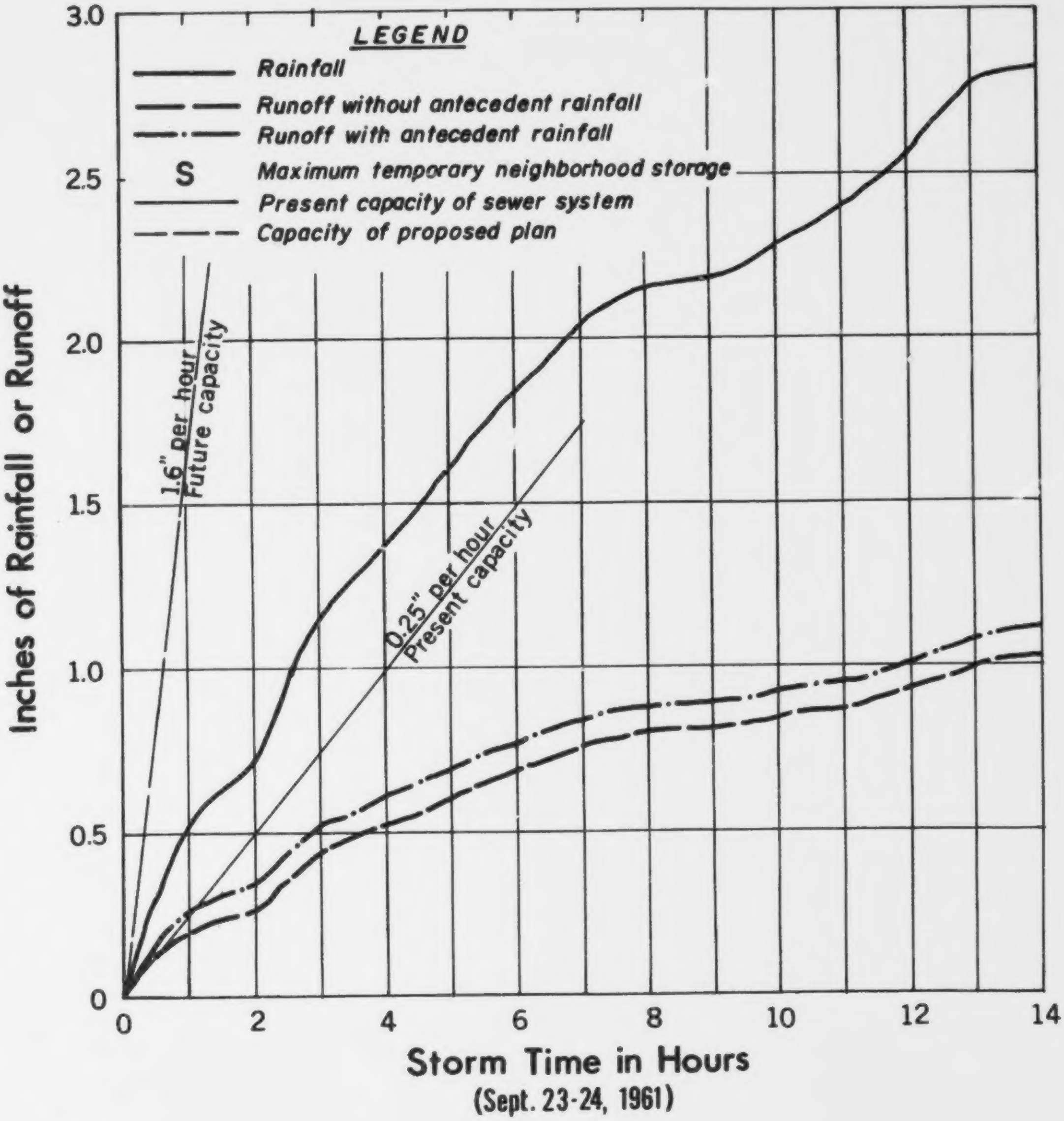


CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

HYETOGRAPH
Sept. 13-14, 1961

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-13

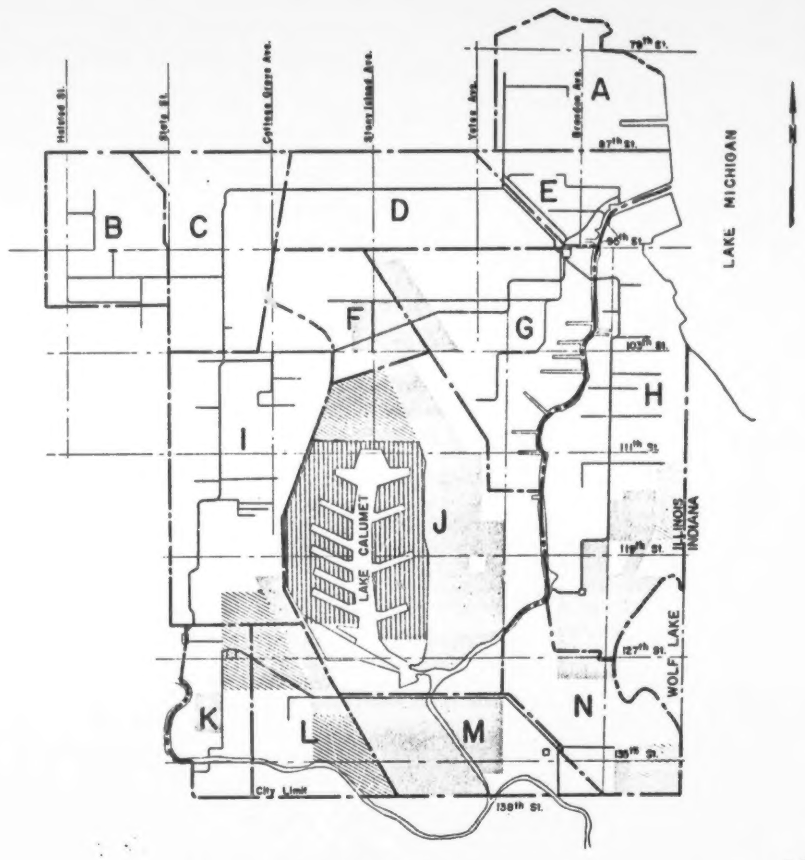


CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

HYETOGRAPH
Sept. 23-24, 1961

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-14


Halsted St.
State St.
Cottage Grove Ave.
Stony Island Ave.
Yates Ave.
Brandon Ave.
79th St.
87th St.
95th St.
103rd St.
111th St.
119th St.
127th St.
135th St.
138th St.
A
B
C
D
E
F
G
H
I
J
K
L
M
N
LAKE CALUMET
LAKE MICHIGAN
WOLF LAKE
ILLINOIS
INDIANA
City Limit
N

CALUMET AREA

LEGEND

- A Area designation
- — · — Area boundary
- ┼ Existing Main and Intercepting sewers
- ☐ Vacant land
- ▥ Potential Lake Calumet Development
- ▨ Unsewered Public Land
- ☐ Sewered residential and other developed areas

EXISTING SEWER OUTLET CAPACITIES			
AREA	ACRES	OUTLET cfs	cfs/Ac
A	1200	200	0.17
B	1150	120	0.10
C	1420	360	0.25
D	1470	220	0.15
E	730	80	0.11
F	800	50	0.06
G	2080	200	0.10
H	2800	500	0.18
I	2400	320	0.13
J	4200	—	—
K	750	60	0.08
L	1050	—	—
M	1200	—	—
N	1150	120	0.10

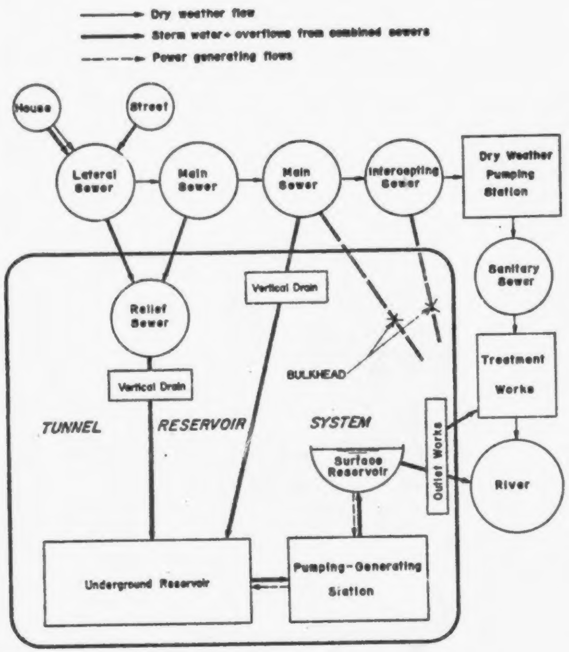
Scale 0 ——— 1 Mile

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

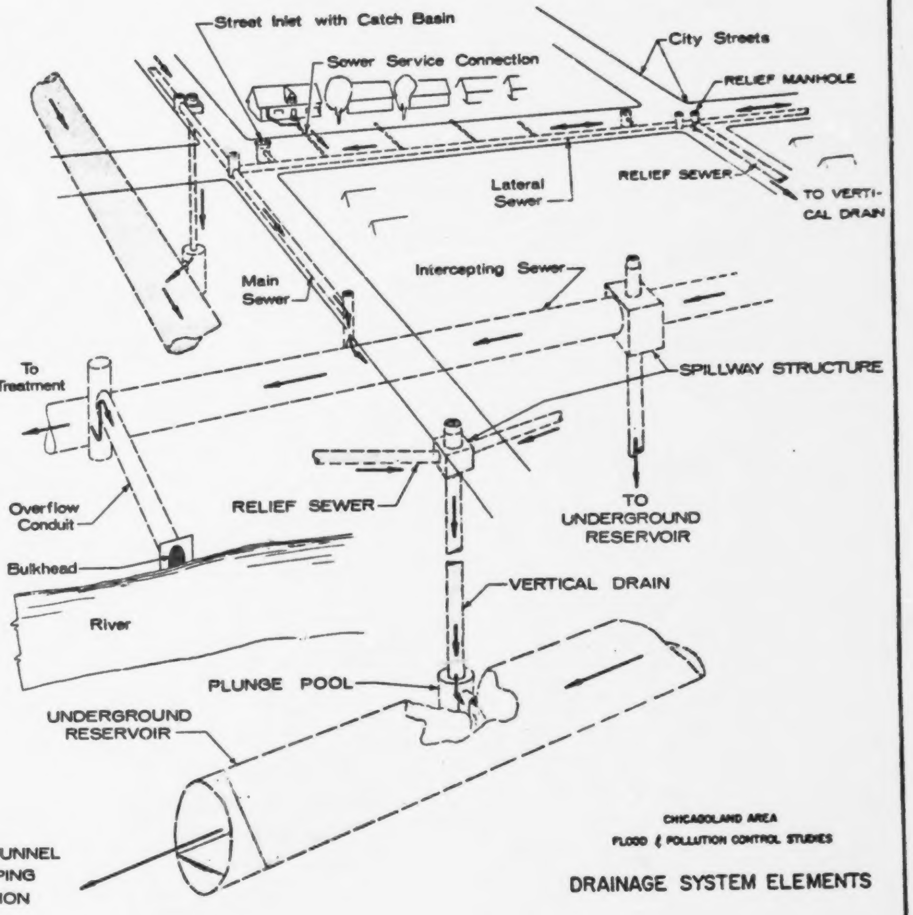
LAND USE AND DRAINAGE BOUNDRIES

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT NO. 6524

EXHIBIT A-15


Dry weather flow
Storm water + overflows from combined sewers
Power generating flows
House
Street
Lateral Sewer
Main Sewer
Main Sewer
Intercepting Sewer
Dry Weather Pumping Station
Sanitary Sewer
Treatment Works
River
Relief Sewer
Vertical Drain
Vertical Drain
BULKHEAD
TUNNEL
RESERVOIR
SYSTEM
Surface Reservoir
Outlet Works
Underground Reservoir
Pumping-Generating Station

FLOW SHEET-PROPOSED SYSTEM

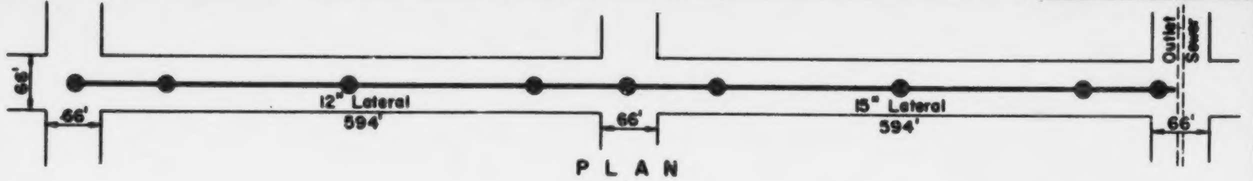

Street Inlet with Catch Basin
Sewer Service Connection
City Streets
RELIEF MANHOLE
RELIEF SEWER
TO VERTICAL DRAIN
Lateral Sewer
Intercepting Sewer
Main Sewer
To Treatment
SPILLWAY STRUCTURE
Overflow Conduit
RELIEF SEWER
TO UNDERGROUND RESERVOIR
Bulkhead
VERTICAL DRAIN
River
PLUNGE POOL
UNDERGROUND RESERVOIR
TO TUNNEL PUMPING STATION

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

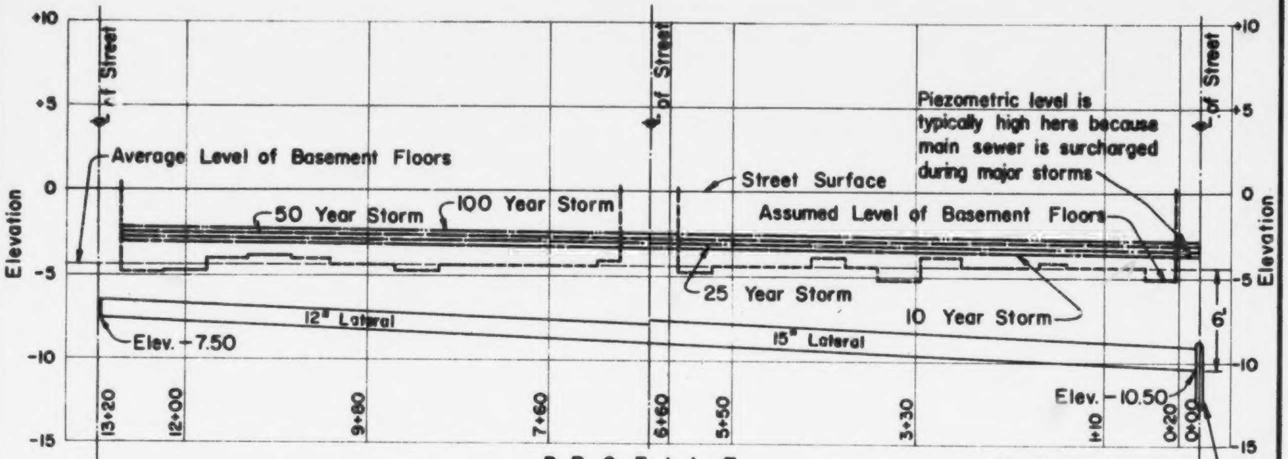
DRAINAGE SYSTEM ELEMENTS

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-16

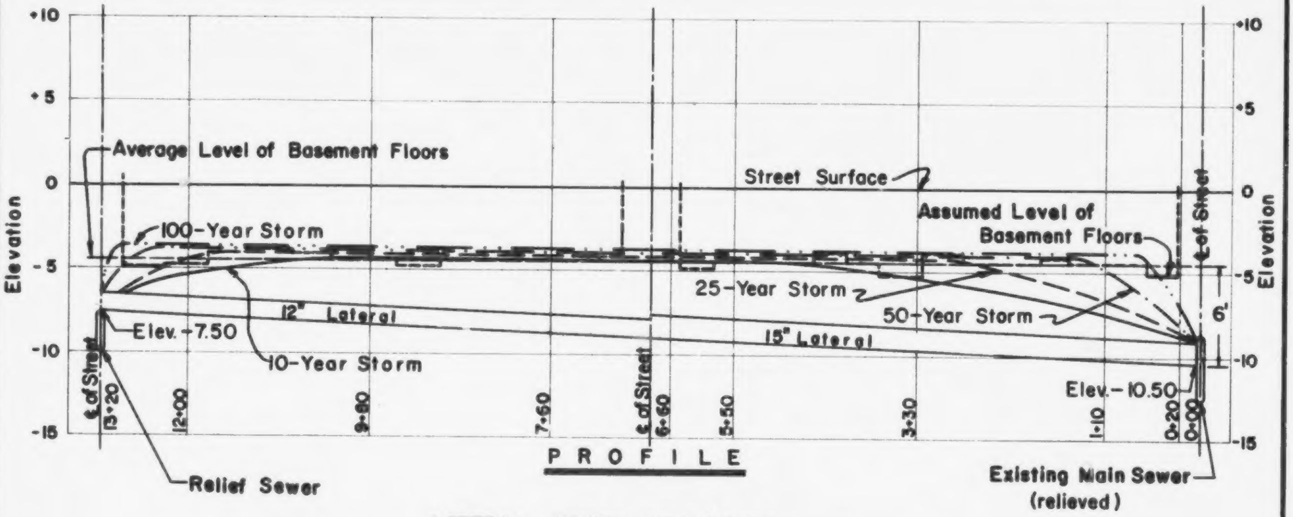

66'
66'
12" Lateral
594'
66'
15" Lateral
594'
66'
Outlet Sewer

PLAN


+10
+5
0
-5
-10
-15
Elevation
℄ of Street
℄ of Street
℄ of Street
Average Level of Basement Floors
Street Surface
Piezometric level is typically high here because main sewer is surcharged during major storms
50 Year Storm
100 Year Storm
Assumed Level of Basement Floors
25 Year Storm
10 Year Storm
12" Lateral
15" Lateral
Elev. -7.50
Elev. -10.50
6'
13+20
12+00
9+80
7+60
6+60
5+50
3+30
1+10
0+20
0+00
Existing Main Sewer

PROFILE

LATERAL SEWER HYDRAULICS
PRESENT PIEZOMETRIC GRADIENTS IN TYPICAL LATERAL


+10
+5
0
-5
-10
-15
Elevation
Average Level of Basement Floors
Street Surface
Assumed Level of Basement Floors
℄ of Street
100-Year Storm
25-Year Storm
50-Year Storm
10-Year Storm
12" Lateral
15" Lateral
Elev. -7.50
Elev. -10.50
6'
℄ of Street
13+20
12+00
9+80
7+60
℄ of Street
6+60
5+50
3+30
1+10
0+20
0+00
Relief Sewer
Existing Main Sewer (relieved)

PROFILE

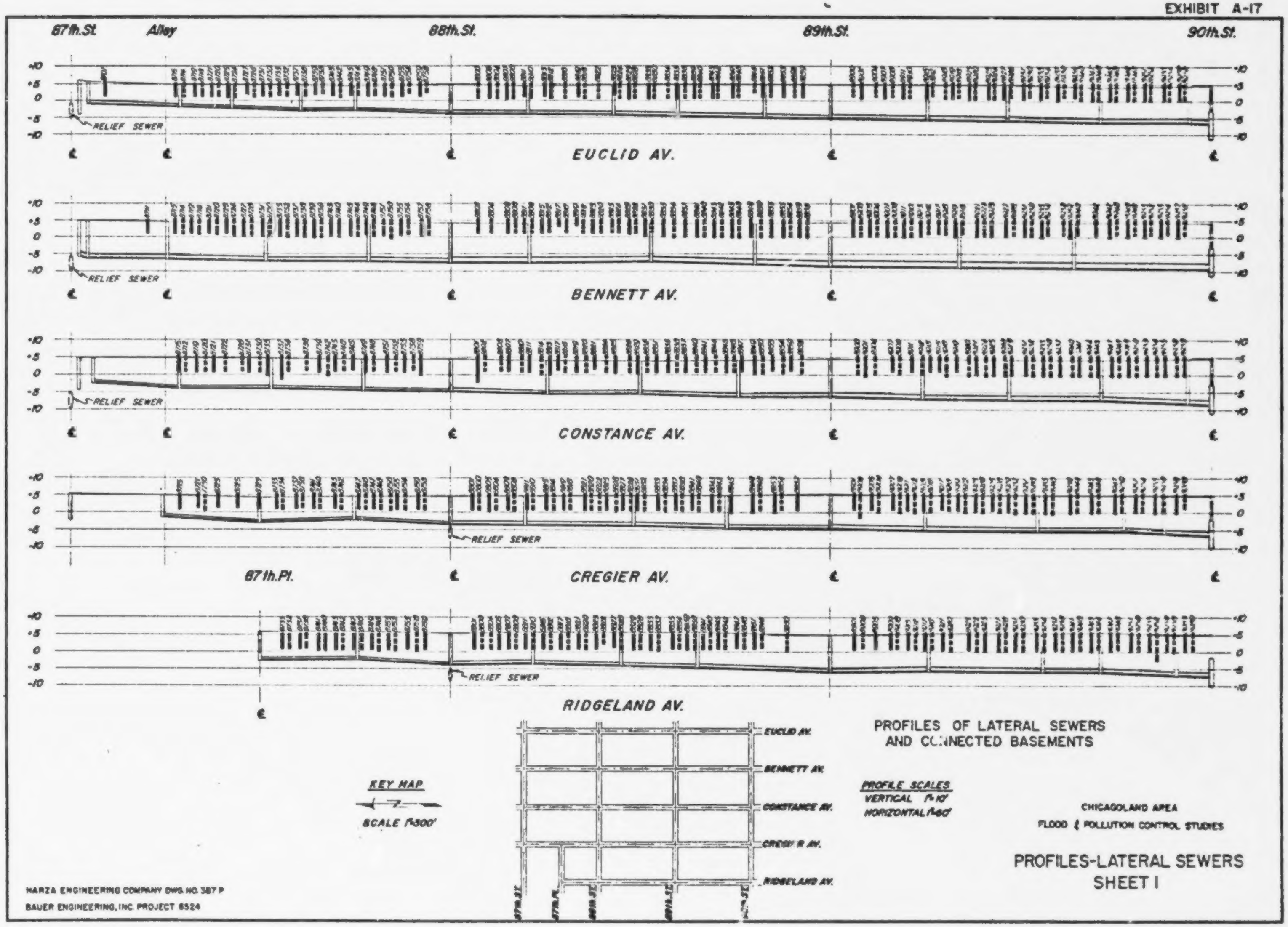
LATERAL SEWER HYDRAULICS
PROPOSED PIEZOMETRIC GRADIENTS IN TYPICAL LATERAL
WITH DEEP TUNNEL

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT NO. 6524

PIEZOMETRIC GRADIENTS IN LATERALS

EXHIBIT A-17


87th.St.
Alley
88th.St.
89th.St.
90th.St.
+10
+5
0
-5
-10
RELIEF SEWER
EUCLID AV.
BENNETT AV.
CONSTANCE AV.
87th.Pl.
CREGIER AV.
RIDGELAND AV.
KEY MAP
SCALE 1"=300'
EUCLID AV.
BENNETT AV.
CONSTANCE AV.
CREGIER AV.
RIDGELAND AV.
PROFILE SCALES
VERTICAL 1"=10'
HORIZONTAL 1"=60'

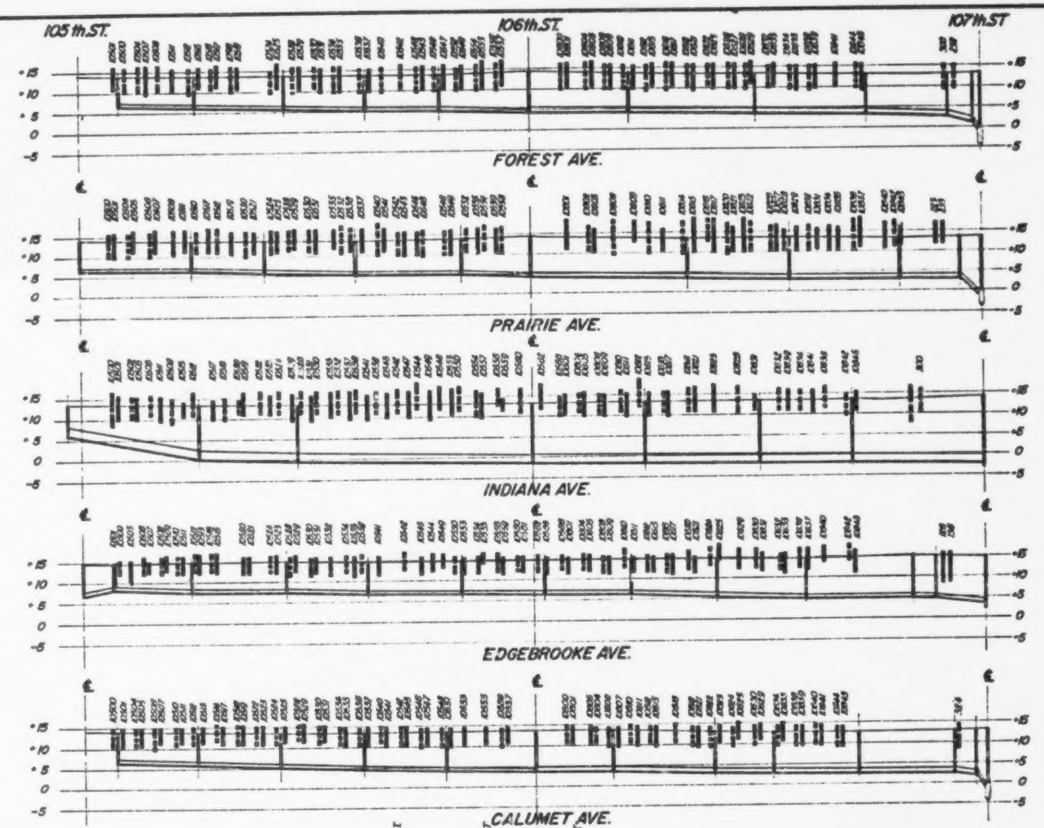
PROFILES OF LATERAL SEWERS
AND CONNECTED BASEMENTS

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

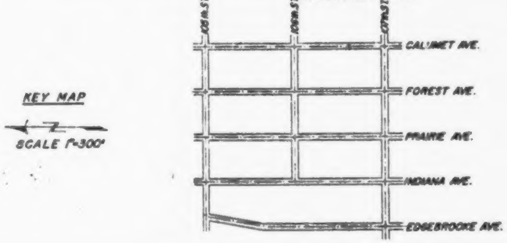
PROFILES-LATERAL SEWERS
SHEET I

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-18


105th ST.
106th ST.
107th ST
+15
+10
+5
0
-5
FOREST AVE.
PRAIRIE AVE.
INDIANA AVE.
EDGEBROOKE AVE.
CALUMET AVE.
℄

KEY MAP


105th ST
106th ST
107th ST
CALUMET AVE.
FOREST AVE.
PRAIRIE AVE.
INDIANA AVE.
EDGEBROOKE AVE.
N
SCALE 1"=300'

PROFILES OF LATERAL SEWERS AND CONNECTED BASEMENTS

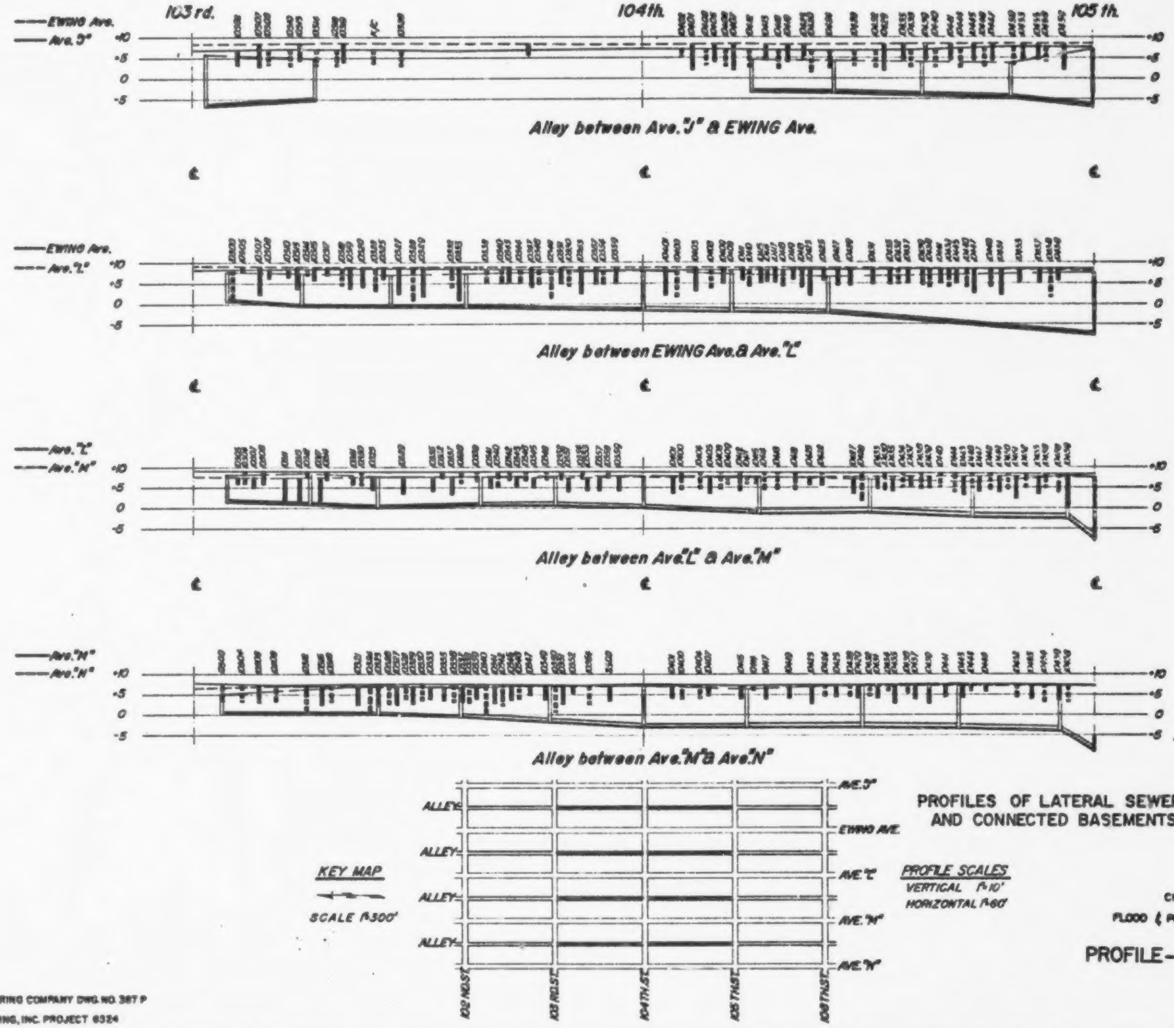
PROFILE SCALES
VERTICAL 1"= 10'
HORIZONTAL 1"= 60'

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

PROFILES-LATERAL SEWERS
SHEET 2

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT 4-19


EWING Ave.
Ave."J"
103rd.
104th.
105th.
+10
+5
0
-5
EWING Ave.
Ave."L"
Ave."L"
Ave."M"
Ave."M"
Ave."N"
KEY MAP
SCALE 1"=300'
ALLEY
AVE."J"
EWING AVE.
AVE."L"
AVE."M"
AVE."N"
103RD.ST.
103RD.ST.
104TH.ST.
105TH.ST.
106TH.ST.

Alley between Ave."J" & EWING Ave.

Alley between EWING Ave. & Ave."L"

Alley between Ave."L" & Ave."M"

Alley between Ave."M" & Ave."N"

PROFILES OF LATERAL SEWERS
AND CONNECTED BASEMENTS

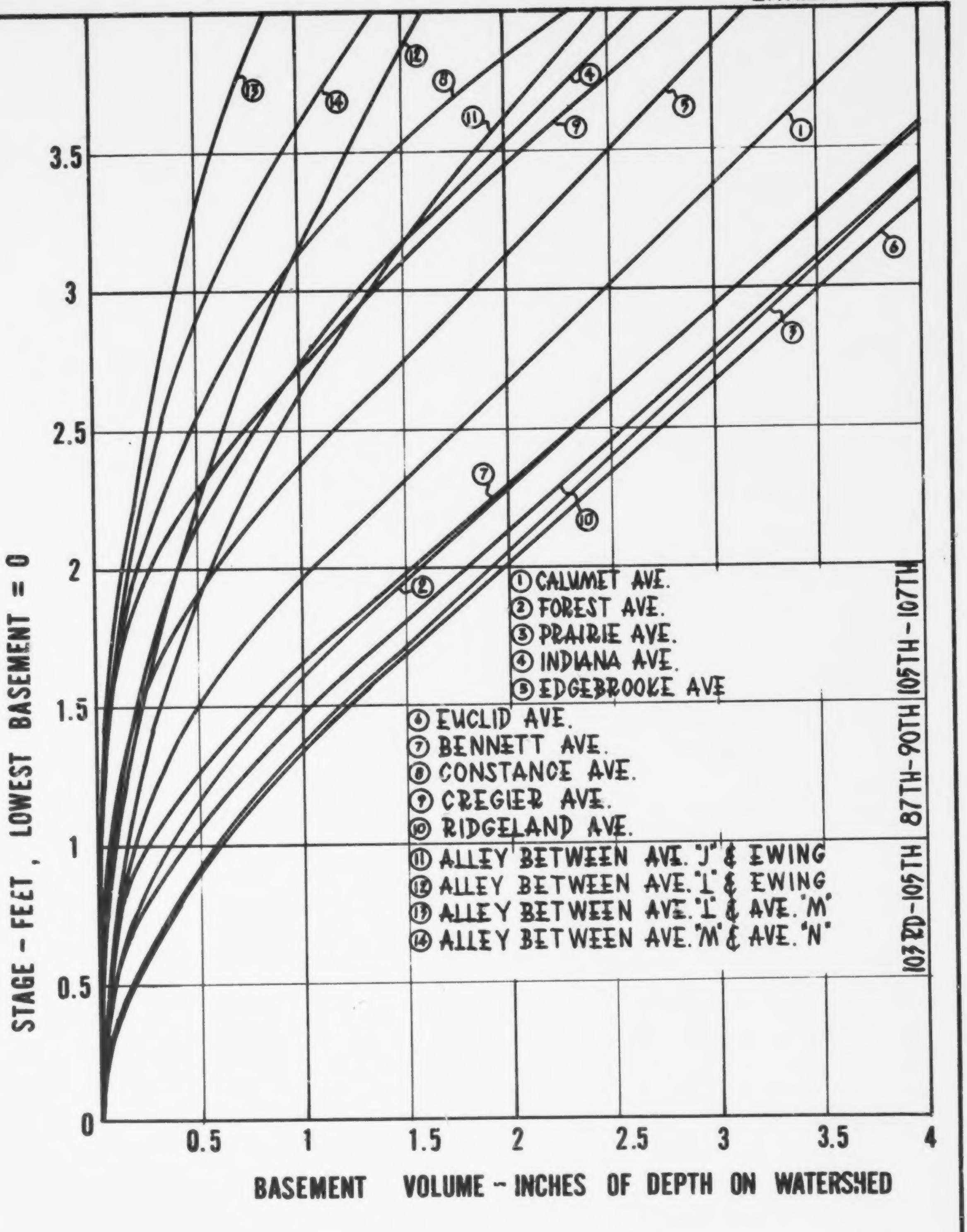
PROFILE SCALES
VERTICAL 1"=10'
HORIZONTAL 1"=60'

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

PROFILE-LATERAL SEWERS
SHEET 3

HARZA ENGINEERING COMPANY DWG.NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6324

EXHIBIT A-20

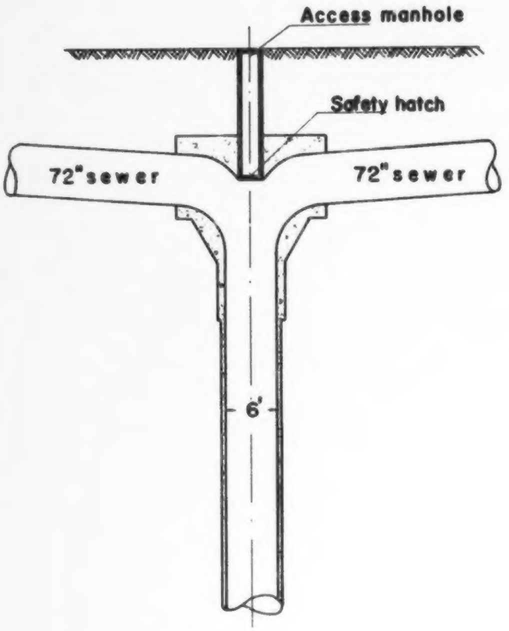


CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

BASEMENT VOLUME-
STAGE RELATIONSHIP

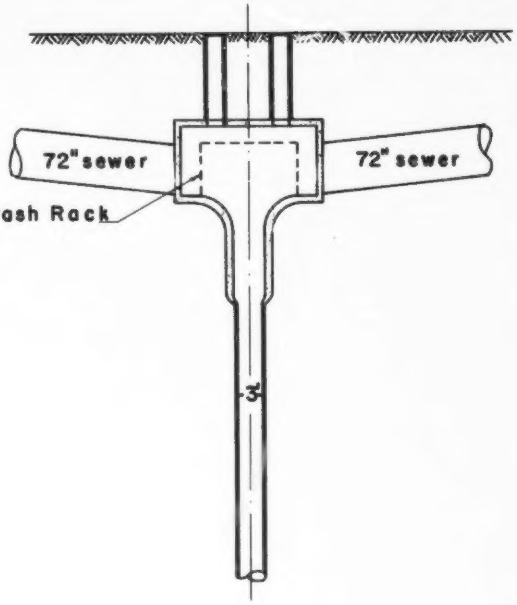
HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A-21


Access manhole
Safety hatch
72" sewer
72" sewer
6'

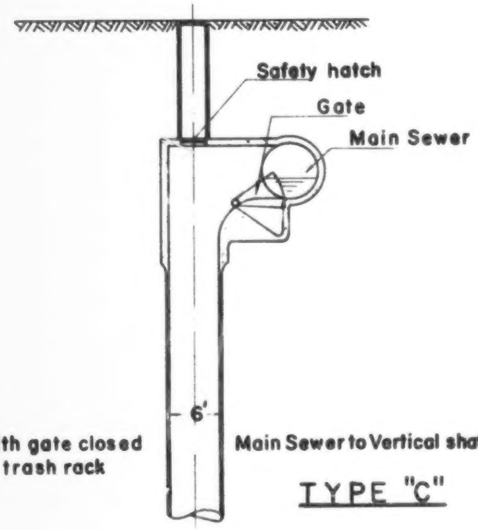
Relief Sewer to Vertical shaft

TYPE "A"


72" sewer
72" sewer
Trash Rack
3'

Relief Sewer to Vertical shaft

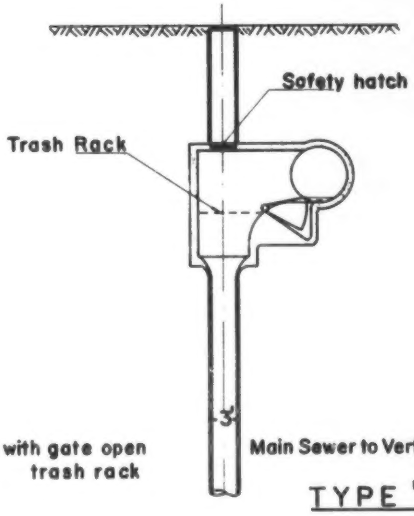
TYPE "B"


Safety hatch
Gate
Main Sewer
6'

Shown with gate closed
without trash rack

Main Sewer to Vertical shaft

TYPE "C"


Safety hatch
Trash Rack
3'

Shown with gate open
with trash rack

Main Sewer to Vertical shaft

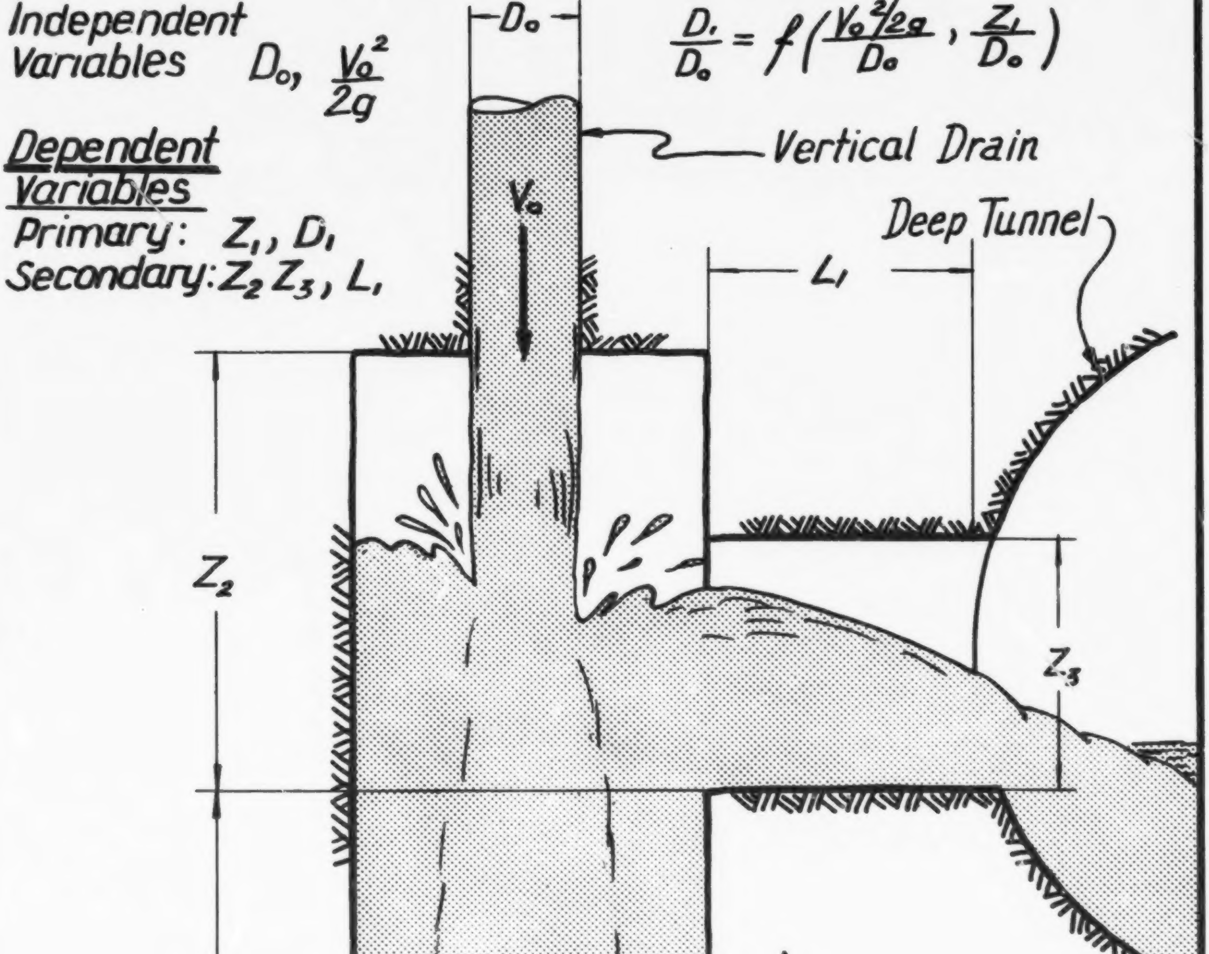
TYPE "D"

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

VERTICAL DRAINS

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT NO. 6524

EXHIBIT A-22


Independent Variables D_o, $\frac{V_o^2}{2g}$
Dependent Variables
Primary: Z_1, D_1
Secondary: Z_2 Z_3, L_1
D_o
$\frac{D_1}{D_o} = f\left(\frac{V_o^2/2g}{D_o}, \frac{Z_1}{D_o}\right)$
Vertical Drain
Deep Tunnel
V_o
L_1
Z_2
Z_3
Z_1
D_1
Galena - Platteville Dolomite

NOTE

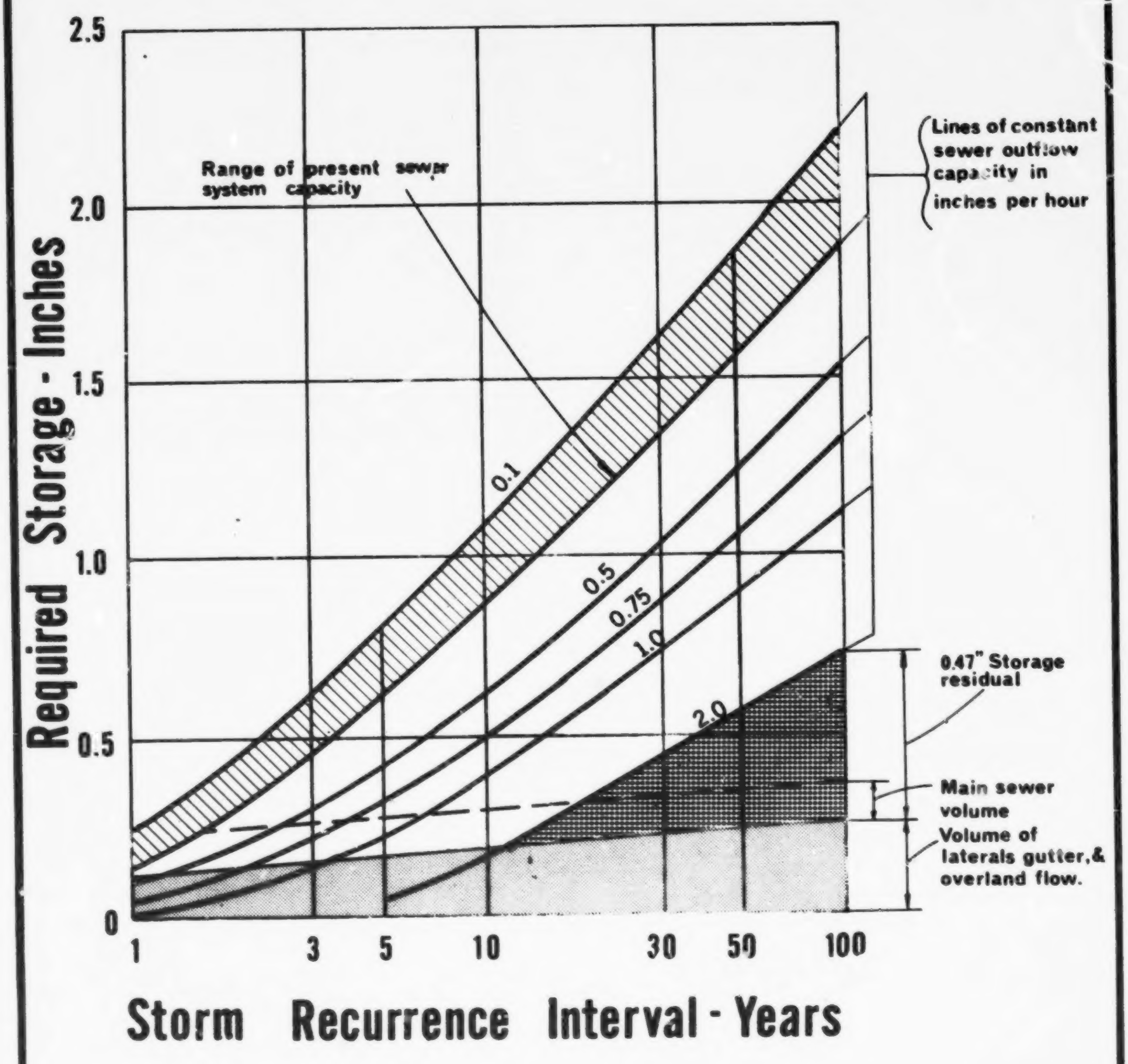
Dimensional parameters shown will be subject of hydraulic model studies.

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

PLUNGE POOL

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT A- 23



CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

NEIGHBORHOOD STORAGE
REQUIREMENTS

HARZA ENGINEERING COMPANY DWG. NO. 387 P
BAUER ENGINEERING, INC. PROJECT 6524

Appendix B

GEOLOGY

Introduction

Purpose and Scope

The geologic investigations were undertaken to aid in the engineering evaluation of the proposed Deep Tunnel Plan. The scope of these studies was limited to a pre-feasibility study.

Previous Studies

The geology of the Chicago Region has been studied by numerous geologists between 1868 and the present date. The results of these studies are available in numerous geologic reports. Most of these reports are very restrictive in scope. The most comprehensive report on the general geology of the area is that by Bretz[1/] published by the Illinois Geological Survey. The most comprehensive report on groundwater is that compiled by the Illinois State Water Survey and the Illinois Geological Survey. [2/]

Present Studies

The present geologic investigations were undertaken by Harza Engineering Company during the period 1965-1966. These investigations

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[1/] "Geology of the Chicago Region," by J. Harlen Bretz. Illinois Geological Survey. 1939.

[2/] "Preliminary Report on Ground-Water Resources of the Chicago Region, Illinois," by Max Suter et al. State Water Survey and State Geological Survey. 1959.

consisted of reviewing existing data, performing field inspections, and consulting with state agency experts. The data reviewed consisted of geologic, groundwater, and industrial minerals reports, and construction records of existing tunnels in the area. Field inspections were made of the Stearns Quarry, within the City limits of Chicago, and exposures in quarries and outcrops in northern Illinois. Discussions with geologic and groundwater experts in private practice in the area and the Illinois State Geological and Water Surveys were most helpful in evaluating the characteristics of the units studied and in directing attention to existing, related data.

Geologic Setting

The proposed underground reservoir is located within the western margin of the Great Lakes Section of the Central Lowland Physiographic Province. This section encloses the eastern four of the Great Lakes; and it is characterized by a poorly integrated drainage, with numerous lesser lakes, lacustrine plains, moraines, and some cuestaform ridges.

With respect to geologic structure the project is located on the north-eastern flank of the Kankakee Arch. The Kankakee Arch, as shown on Exhibit B-1, extends across northern Indiana and northeastern Illinois, connecting the Cincinnati Arch with the Wisconsin Arch. These structural features separate the Illinois Basin from the Michigan Basin which lies south of the Canadian Shield.

The oldest rocks are the Pre-Cambrian igneous and metamorphic rocks of the Canadian Shield. These rocks outcrop north of the Great Lakes and extend southward into central Wisconsin. Overlying the eroded surface of the Pre-Cambrian rocks are marine sediments of the Paleozoic era consisting of limestones, dolomites, shales, and sandstones. They form the stratigraphic section in the Illinois and Michigan Basins and are exposed on the flanks of the arch separating these two basins. The stratigraphic sequence is shown on the geologic section on Exhibit B-1. The proposed underground reservoir would be located in the Paleozoic rocks of the Ordovician and Silurian Systems.

Glacial deposits cover most of the area. The deposits consist of morainal ridges parallel to the present lake shore of Lake Michigan. The areas between the ridges are level and covered with material deposited as ground moraine and out wash deposits. Lacustrine deposits corresponding to three beach levels have been mapped around southern Lake Michigan.

Geology of the Project Area

Areal Geology

The areal distribution of the bedrock units beneath the glacial drift in northeastern Illinois and the Chicago area is shown on Exhibit B-2. Only the strata of the Ordovician and Silurian Periods are exposed in northeastern Illinois, and the bedrock exposed in the Chicago area is restricted to the Silurian. These rocks are covered in most of the area by the glacial drift of the Pleistocene Period.

Stratigraphic Sequence

General. The geologic sequence of the stratigraphic units in the study area is shown on the Generalized Geologic Column on Exhibit B-3. The upper 2000 feet of the column has been developed from well log data. The lower portion of the column, the strata of Cambrian age, is inferred from regional data. Only the geologic groups and formations of the Ordovician Period and younger affect the Plan. These groups and formations are described below.

Prairie du Chien. The Prairie du Chien Group varies in thickness from 0 to 300 feet. This group is composed almost entirely of finely to coarsely crystalline, cherty, white to light grey to pink dolomite with lenses of sandstone. Before deposition of the overlying sandstone, this unit was subjected to severe erosion and weathering. In some areas the entire unit has been removed by erosion.

Ancell. The Ancell Group is composed of the Glenwood Formation and St. Peter sandstone. This group lies unconformably on the very irregular erosional surface and, therefore, varies in thickness from 130 to 650 feet. Most of this group is a fine-to-medium-grained, incoherent to friable sandstone with varying amounts of silt.

Platteville. The Platteville Group overlies the Glenwood sandstone and is divided into four formations: the Pecatonic dolomite, Mifflin formation, Grand Detour formation, and the Nachusa formation. The Pecatonia is fine-grained, brownish dolomite with some thin argillaceous partings. The Mifflin is a light grey dolomite with some shale partings. The Grand Detour is a grey fine-grained dolomite with some chert and shale partings. The Nachusa consists of buff medium-grained dolomite that grades into limestone. The shales in the Platteville in western Illinois grade to dolomite in the Chicago area, making it difficult to differentiate between the formations in this group in this area.

Galena. The Galena Group unconformably overlies the Platteville Group and is composed of the Guttenberg formation and the Wise Lake-Dunleith formations. The Guttenberg formation is a buff, medium-grained dolomite that grades to fine-grained limestone. The Wise Lake-Dunleith formations are buff, fine to medium grained dolomites with scattered chert nodules. The formations of the Galena are massive and uniform over a large area.

Maquoketa. The Maquoketa Group is over 100 feet in thickness and can be divided into four formations. The lower Scales formation is a brittle, dark brown dolomitic shale grading locally to dark brown, argillaceous dolomite. The middle Ft. Atkinson formation is dominantly brown to grey, fine-to-coarse-grained, fossiliferous, argillaceous, speckled dolomite and limestone. The upper shale formation, the Brainard, is a greenish grey, weak, silty, dolomitic shale that grades into very argillaceous, greenish-grey to grey dolomite. The overlying Neda formation is only of local occurrence and is a thin hematitic, oolitic shale.

Alexandrian. The Alexandrian Group consists of two formations: the Edgewood below and the Kankakee above. Both formations are light grey to buff, cherty, finely crystalline dolomites. The group varies in thickness from 20 to 100 feet.

Niagaran. The Niagaran Group is white to light grey, finely to medium crystalline, compact dolomite with varying amounts of silt. White to light grey chert generally occurs in the upper part. At the base, the dolomite is commonly green, pink, or red and is slightly silty. This group is over 300 feet in thickness. The Niagaran Group has undergone a period of erosion with the result that the rock surface is very irregular. Some solution has occurred, generally along joints.

Pleistocene. The proposed underground reservoir lies within the limits formerly occupied by glacial Lake Chicago. The surface deposits are generally lacustrine sands from a few to 10 feet in thickness. Locally, these sands are overlain by organic swamp deposits. Underlying the sand deposits are clay deposits representing the ground moraines of the various glacial advances. The clays are stiff to hard, increasing in degree of compactness with depth. The thickness of the glacial deposits varies from negligible to over 100 feet in the Chicago area due to irregularities in the bedrock surface. Within the immediate study area, it appears the glacial deposits do not exceed about 75 feet in thickness.

The base of the Pleistocene deposits is shown by the inferred bedrock topography on Exhibit B-4. An inspection of this exhibit shows several buried easterly trending bedrock valleys cutting entirely across the project area. The valleys apparently exceed 50 feet in depth from the adjacent rock surface. The lowest portions of the valleys are often filled with permeable glacial deposits; and these materials, therefore, may be present in the project area also. Local deep glacial scouring of the bedrock surface may have also occurred.

Structural Geology

The study area is located on the northeastern flank of the Kankakee Arch and within the southwestern margin of the Michigan Basin. The strata have a NW-SE strike and a northeasterly dip of 10 to 20 feet per mile. Structural contours drawn on the base of the Platteville Group and the top of the Galena Group are shown on Exhibit B-5. Structural contours drawn on the base of the Niagaran strata are shown on Exhibit B-4.

A major structural break, the Sandwich Fault, exists southwest of the study area. No faulting is known to exist within the Calumet area, but minor faults and local variations in the dip may exist.

Geology of Civil Works

The influence of the geology on the overall feasibility of the proposed development has been thoroughly investigated. The investigation has been largely directed toward the selection of a stratigraphic unit suitable for the development of the proposed Deep Tunnel Plan. Consideration has also been given to the geologic conditions that will be encountered in the auxiliary structures, including the shafts, ,the vertical drains, and the surface reservoir.

Underground Reservoir

The criteria for the selection of a geologic unit suitable for the underground reservoir are: (1) It must not require lining except in isolated zones; (2) it must not require support other than rock bolts except in the isolated areas requiring linings; and (3) it must be below the piezometric surface and yet have no large inflows of water during construction. The strata of the Galena and Platteville Groups are the only ones that meet these criteria. Of the two, the Galena Group is superior. As much of the development as possible should be restricted to this unit.

The Galena Group is massive and does not vary appreciably in thickness or quality over the area. It is considered the best rock for tunneling in the area and will require the least support to maintain long-term stability under the operating conditions of the project.

The Platteville Group is composed of relatively thin dolomites with some shale partings. It is reported that the Platteville grades to dolomite in the Chicago region and that the thin shale partings become less prominent resulting in a more massive rock. If later exploration confirms this, it may be possible to locate more of the underground openings in the Platteville Group.

Both the Galena and Platteville lie below the present piezometric level in the project area. Recharge facilities will be required during the operation of the underground reservoir to maintain the piezometric level above the elevation of the reservoir.[3/] The groundwater occurs only in joints and fissures in the Galena and Platteville, and large inflows of water are not expected during construction.

Shafts and Vertical Drains

The shafts will penetrate the glacial drift, the Silurian dolomites, the Maquoketa shale and enter the Galena.

The portion of the shafts excavated in the glacial drift will require structural support. It is recommended that the shafts be located to avoid the buried valleys in the bedrock surface where large quantities of groundwater may be encountered. The portion of shafts excavated in shale may also require structural support. The entire shaft will require lining to prevent groundwater inflow. The vertical drains will require casing cemented to the surrounding rock for the same reasons.

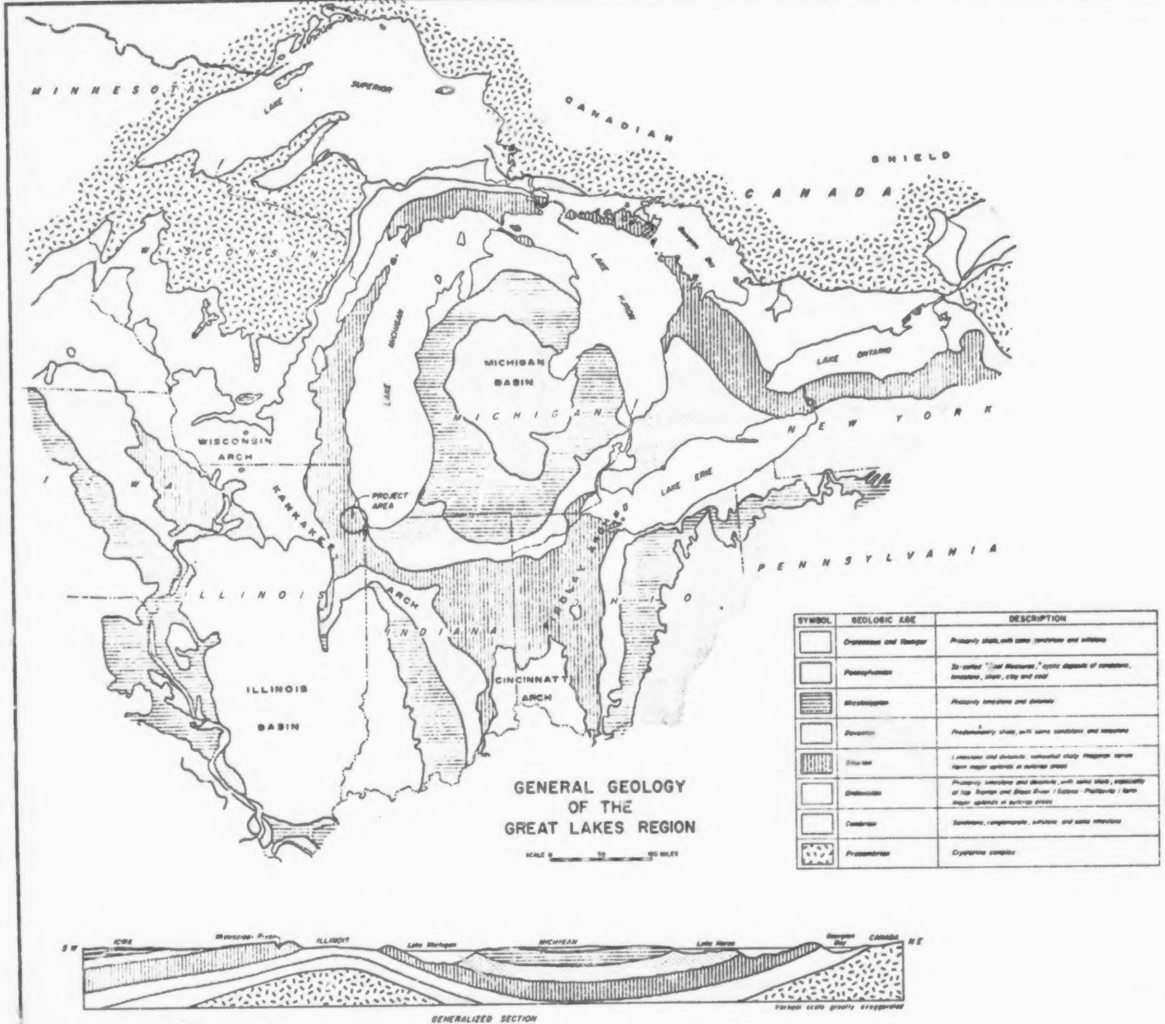
Surface Reservoir

The surface reservoir is located in a swampy area formerly occupied by ancient Lake Chicago. Surface materials consist of organic swamp deposits, estimated to be up to three feet in depth, and underlying lacustrine sands ranging from three to five feet in thickness. Underlying the lacustine sands are stiff glacial clays.

[3/] See Appendix D.

Leakage is not expected to be appreciable from an artificial reservoir at this site. Structures can be founded on the sand or underlying clay; but, to limit leakage from the reservoir, a cutoff should extend through the sand to the underlying stiff glacial clay. These clays are relatively impermeable.

EXHIBIT B-1


MINNESOTA
LAKE SUPERIOR
CANADIAN SHIELD
CANADA
WISCONSIN
LAKE MICHIGAN
LAKE HURON
Georgian Bay
LAKE ONTARIO
MICHIGAN BASIN
MICHIGAN
NEW YORK
WISCONSIN ARCH
IOWA
KANKAKEE ARCH
PROJECT AREA
LAKE ERIE
FINDLAY ARCH
PENNSYLVANIA
ILLINOIS
INDIANA
OHIO
CINCINNATI ARCH
ILLINOIS BASIN
SCALE 0 50 100 MILES
SW
IOWA
Mississippi River
ILLINOIS
Lake Michigan
MICHIGAN
Lake Huron
Georgian Bay
CANADA
NE
Vertical scale greatly exaggerated

GENERAL GEOLOGY
OF THE
GREAT LAKES REGION

GENERALIZED SECTION

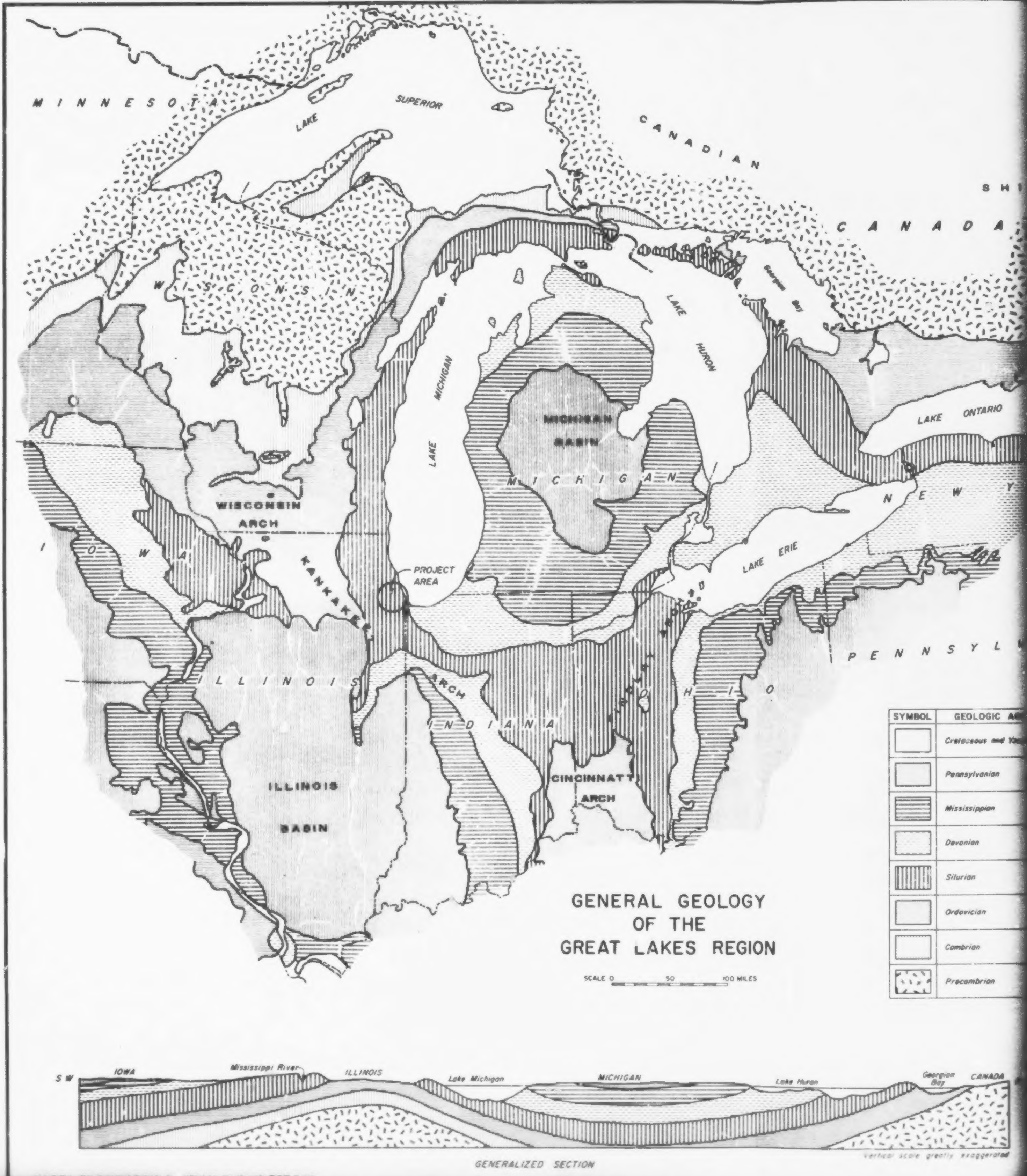
SYMBOL	GEOLOGIC AGE	DESCRIPTION
☐	Cretaceous and Younger	Primarily shale, with some sandstone and siltstone
☐	Pennsylvanian	So-called "Coal Measures," cyclic deposits of sandstone, limestone, shale, clay and coal
▤	Mississippian	Primarily limestone and dolomite
☐	Devonian	Predominantly shale, with some sandstone and limestone
▥	Silurian	Limestone and dolomite, somewhat shaly. Niagaran series form major uplands in outcrop areas
☐	Ordovician	Primarily limestone and dolomite, with some shale, especially at top. Trenton and Black River (Galena - Platteville) form major uplands in outcrop areas
☐	Cambrian	Sandstone, conglomerate, siltstone and sandy limestone
▨	Precambrian	Crystalline complex

NOTE:
1. Map after Geologic Map of the United States, by A. K. Lobeck

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

REGIONAL GEOLOGY

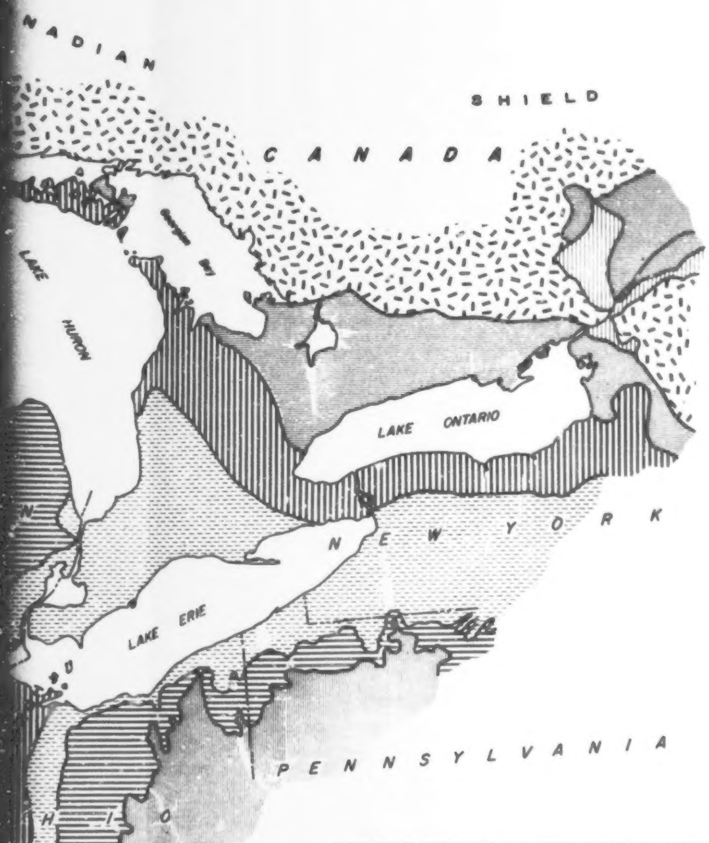
HARZA ENGINEERING COMPANY DWG NO 387 P 21
BAUER ENGINEERING, INC. PROJECT 6524


MINNESOTA
LAKE SUPERIOR
CANADIAN SHI
CANADA
WISCONSIN
LAKE MICHIGAN
LAKE HURON
Georgian Bay
LAKE ONTARIO
MICHIGAN BASIN
MICHIGAN
NEW Y
WISCONSIN ARCH
IOWA
KANKAKEE ARCH
PROJECT AREA
LAKE ERIE
FINDLAY ARCH
PENNSYL
ILLINOIS
INDIANA
OHIO
ILLINOIS BASIN
CINCINNATI ARCH
SYMBOL
GEOLOGIC AGE
Cretaceous and Ter
Pennsylvanian
Mississippian
Devonian
Silurian
Ordovician
Cambrian
Precambrian
SCALE 0 50 100 MILES
SW
IOWA
Mississippi River
ILLINOIS
Lake Michigan
MICHIGAN
Lake Huron
Georgian Bay
CANADA
Vertical scale greatly exaggerated
GENERALIZED SECTION

GENERAL GEOLOGY
OF THE
GREAT LAKES REGION

HARZA ENGINEERING COMPANY DWG. NO. 387 P 2

EXHIBIT B-1


NADIAN
SHIELD
CANADA
LAKE HURON
Georgian Bay
LAKE ONTARIO
LAKE ERIE
NEW YORK
PENNSYLVANIA
HIO

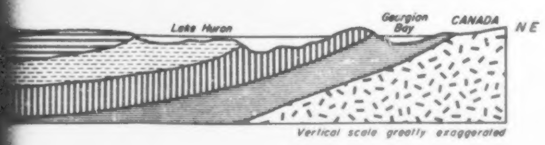
SYMBOL	GEOLOGIC AGE	DESCRIPTION
□	*Cretaceous and Younger*	*Primarily shale, with some sandstone and siltstone.*
▒	*Pennsylvanian*	*So-called "Coal Measures;" cyclic deposits of sandstone, limestone, shale, clay and coal.*
▤	*Mississippian*	*Primarily limestone and dolomite.*
░	*Devonian*	*Predominantly shale, with some sandstone and limestone.*
▥	*Silurian*	*Limestone and dolomite, somewhat shaly. Niagaran series form major uplands in outcrop areas.*
▒	*Ordovician*	*Primarily limestone and dolomite, with some shale, especially at top. Trenton and Black River (Galena - Platteville) form major uplands in outcrop areas.*
□	*Cambrian*	*Sandstone, conglomerate, siltstone, and some limestone.*
⁙	*Precambrian*	*Crystalline complex.*

GEOLOGY
THE
KES REGION

50 100 MILES

NOTE:

1. *Map after Geologic Map of the United States, by A. K. Lobeck.*

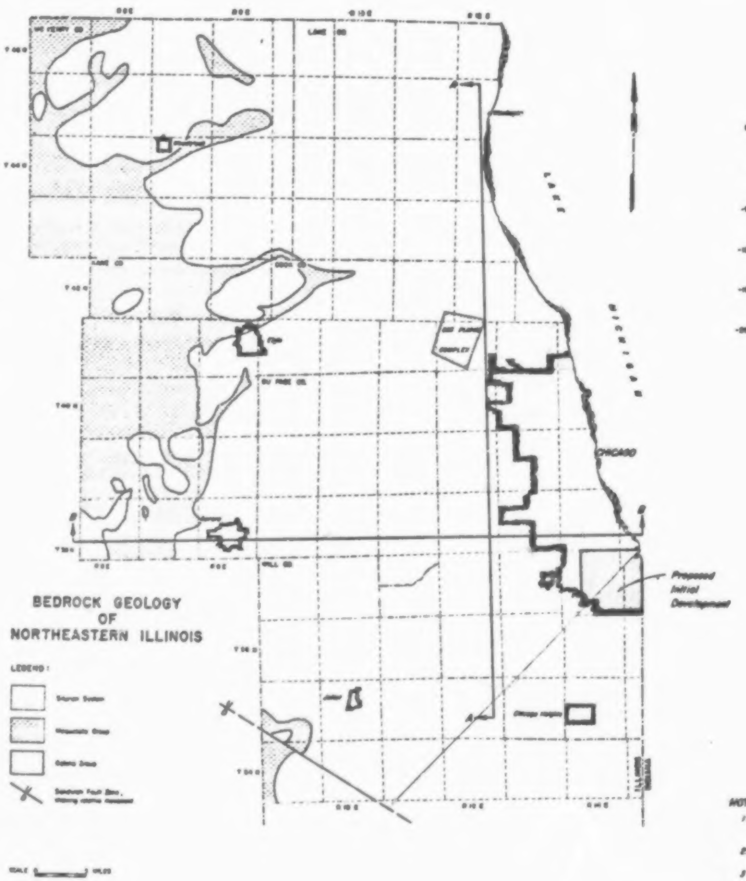

Lake Huron
Georgian Bay
CANADA
NE

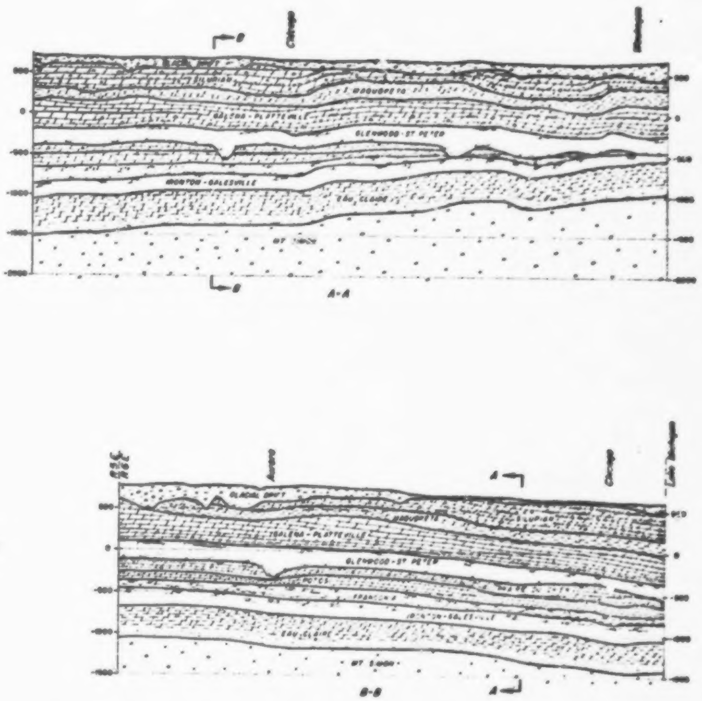
Vertical scale greatly exaggerated

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

REGIONAL GEOLOGY

EXHIBIT B-2


MC HENRY CO
LAKE CO
KANE CO
COOK CO
DU PAGE CO
WILL CO
LAKE
MICHIGAN
CHICAGO
Proposed Initial Development
Chicago Heights
BEDROCK GEOLOGY OF NORTHEASTERN ILLINOIS
LEGEND:
Silurian System
Maquoketa Group
Galena Group
Sandwich Fault Zone, Showing relative movement
SCALE MILES


A-A
B-B
Chicago
GLACIAL DRIFT
MAQUOKETA
GALENA-PLATTEVILLE
GLENWOOD-ST PETER
IRONTON-GALESVILLE
EAU CLAIRE
MT SIMON
500
0
-500
-1000
-1500
-2000

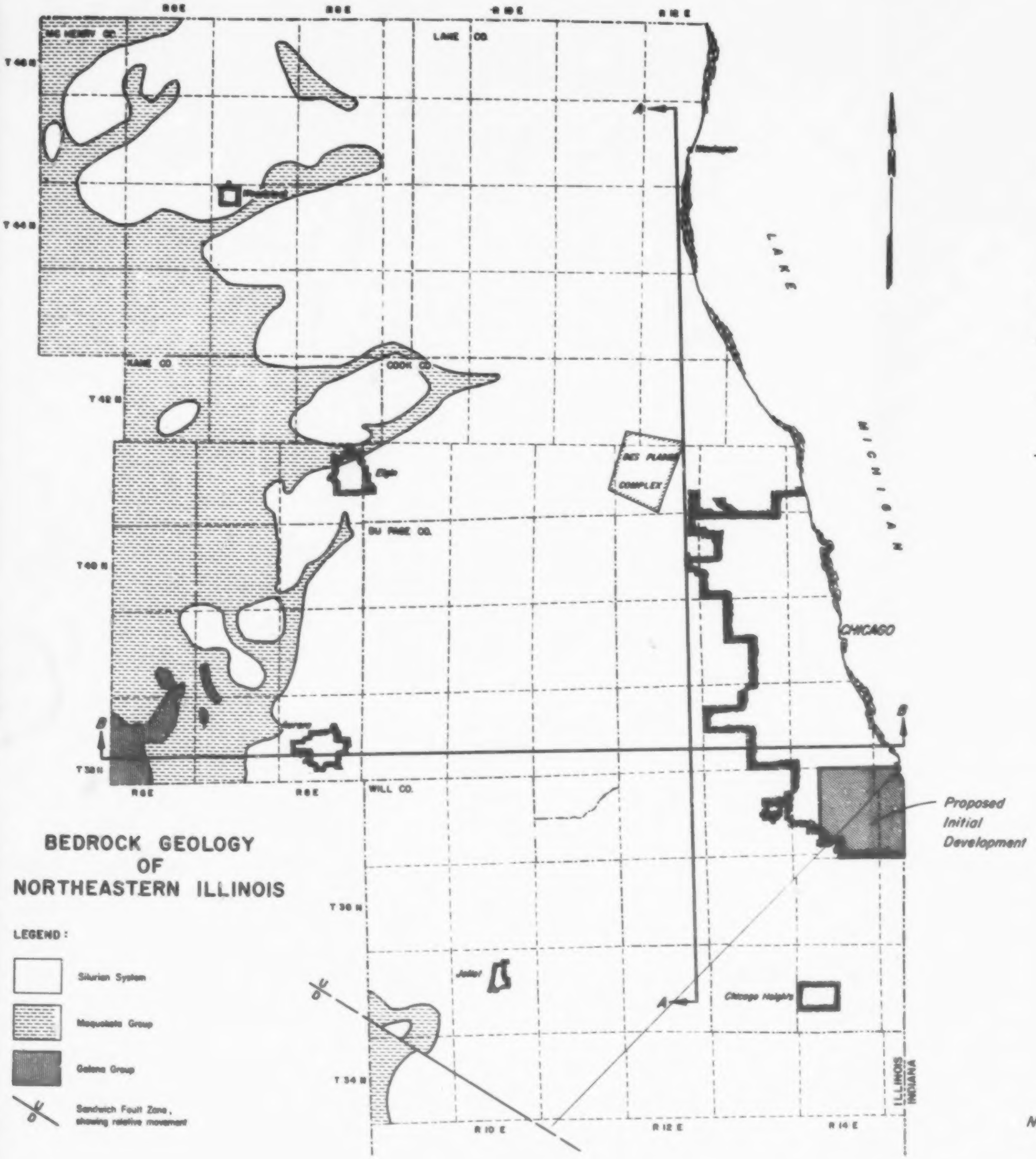
NOTES:

1. Basis for geologic map from Cooperative Ground-water Report 1, Preliminary Report on Ground-water Resources of the Chicago Region, Illinois. Illinois State Water Survey, and Illinois State Geological Survey, 1959
2. For descriptions of geologic units and symbols used on sections, see Exhibit B-3, Generalized Geologic Column
3. Vertical exaggeration of sections approximately 40 x

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

GEOLOGIC MAP AND SECTIONS

HARZA ENGINEERING COMPANY DWG. NO. 367 P
BAUER ENGINEERING, INC. PROJECT 6524

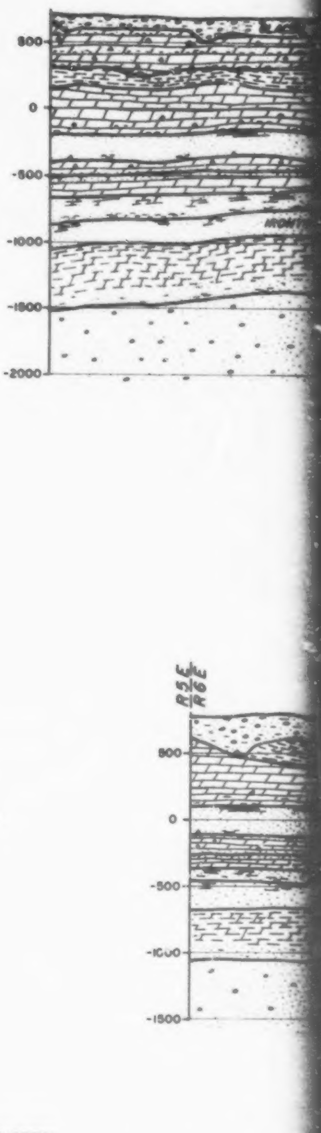

R 6 E
R 8 E
R 10 E
R 12 E
MC HENRY CO.
LAKE CO.
T 46 N
T 44 N
KANE CO.
COOK CO.
T 42 N
Elgin
DES PLAINES COMPLEX
DU PAGE CO.
T 40 N
LAKE
MICHIGAN
CHICAGO
Aurora
B
B
A
A
T 38 N
WILL CO.
T 36 N
Joliet
Chicago Heights
T 34 N
U
D
ILLINOIS
INDIANA
R 10 E
R 12 E
R 14 E
Proposed Initial Development

BEDROCK GEOLOGY
OF
NORTHEASTERN ILLINOIS

LEGEND :

☐	Silurian System
▤	Maquoketa Group
■	Galena Group
⟋ U D	Sandwich Fault Zone, showing relative movement

SCALE 0 5 MILES


500
0
-500
-1000
-1500
-2000
R 5 E
R 6 E
500
0
-500
-1000
-1500

NOTES :

1. *Data for geologic map from Cooperative Grou*
   Ground-water Resources of the Chicago Regi
   Illinois State Geological Survey, 1959
2. *For descriptions of geologic units and symbo*
   Generalized Geologic Column
3. *Vertical exaggeration of sections approximately*

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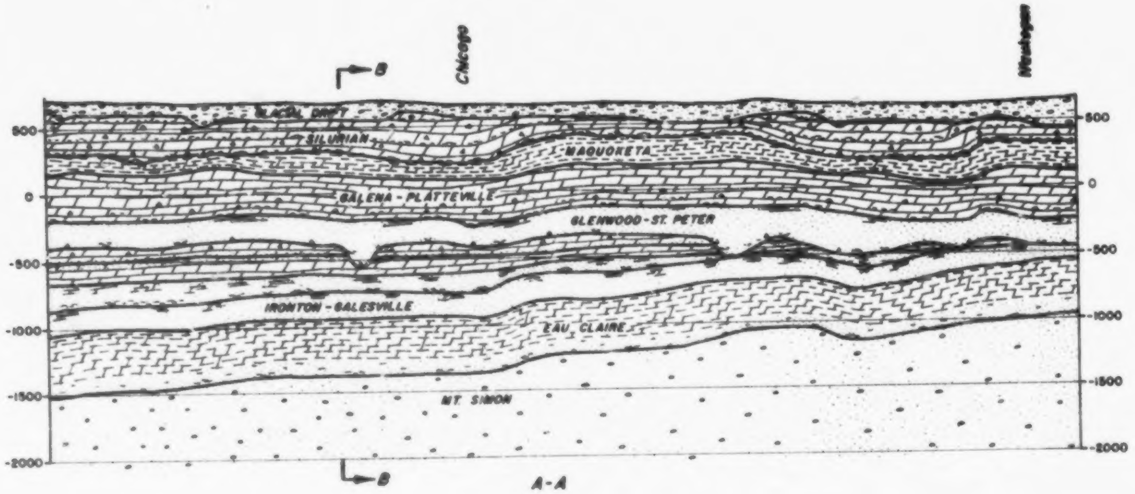
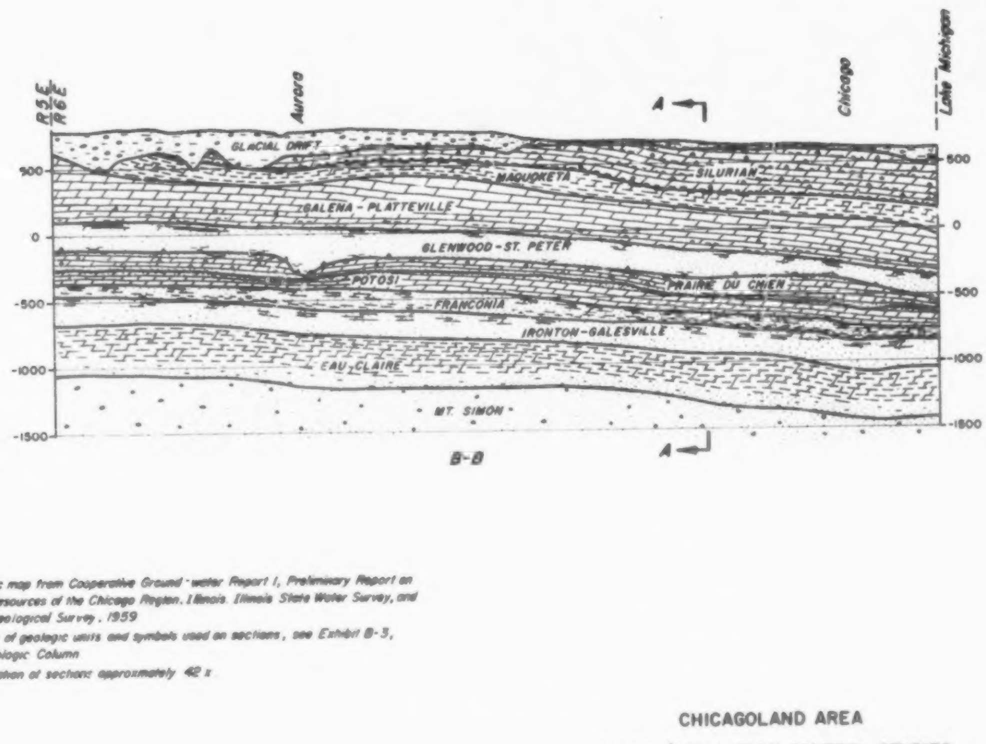
EXHIBIT B-2.

MICHIGAN
CHICAGO
B
Proposed Initial Development
ILLINOIS
INDIANA


B
Chicago
Waukegan
GLACIAL DRIFT
SILURIAN
MAQUOKETA
GALENA - PLATTEVILLE
GLENWOOD - ST. PETER
IRONTON - GALESVILLE
EAU CLAIRE
MT. SIMON
500
0
-500
-1000
-1500
-2000
B

A-A


R5E
R6E
Aurora
A
Chicago
Lake Michigan
GLACIAL DRIFT
MAQUOKETA
SILURIAN
GALENA - PLATTEVILLE
GLENWOOD - ST. PETER
POTOSI
PRAIRIE DU CHIEN
FRANCONIA
IRONTON - GALESVILLE
EAU CLAIRE
MT. SIMON
500
0
-500
-1000
-1500
A

B-B

NOTES:

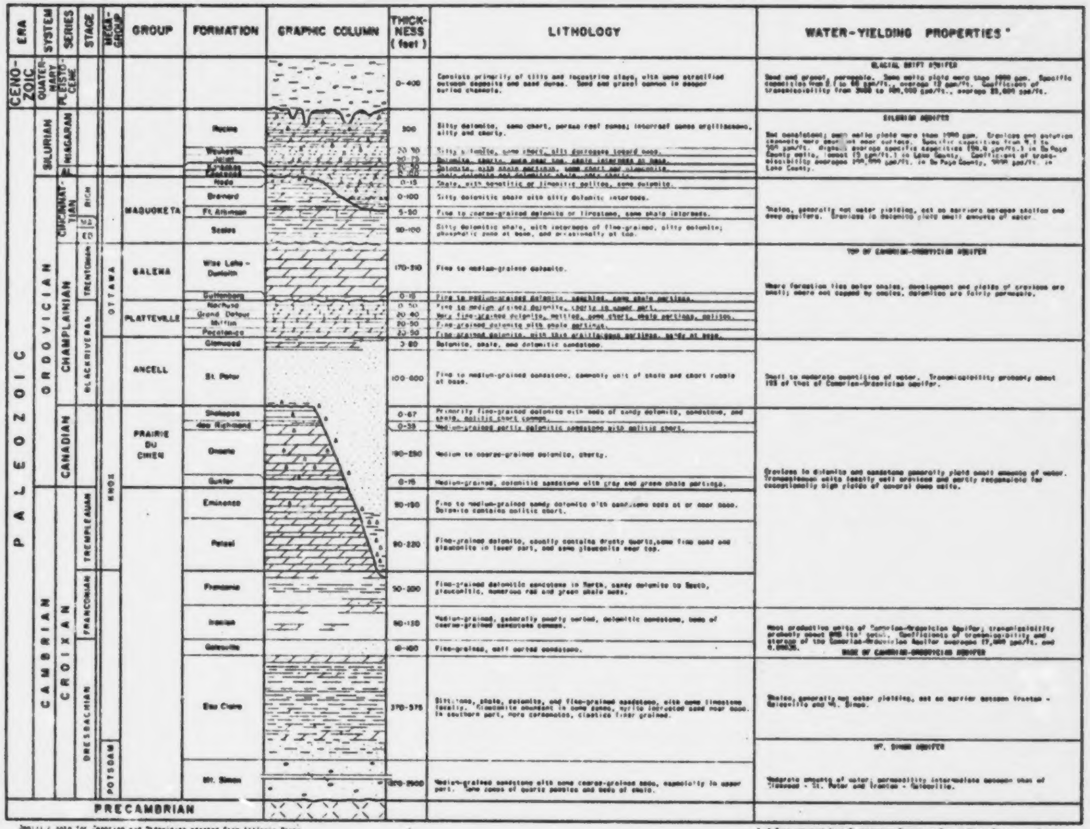
1. *Data for geologic map from Cooperative Ground-water Report I, Preliminary Report on Ground-water Resources of the Chicago Region, Illinois. Illinois State Water Survey, and Illinois State Geological Survey, 1959*
2. *For descriptions of geologic units and symbols used on sections, see Exhibit B-3, Generalized Geologic Column*
3. *Vertical exaggeration of sections approximately 42 x*

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

GEOLOGIC MAP AND SECTIONS

EXHIBIT B-3

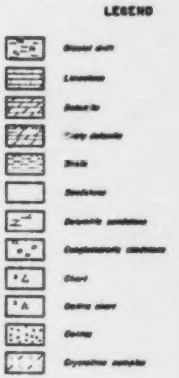
GENERALIZED GEOLOGIC COLUMN



ERA	SYSTEM	SERIES	STAGE	MEGA-GROUP	GROUP	FORMATION	GRAPHIC COLUMN	THICKNESS (feet)	LITHOLOGY	WATER-YIELDING PROPERTIES*
CENOZOIC	QUATERNARY	PLEISTOCENE						0-400	Consists primarily of tills and lacustrine clays, with some stratified outwash deposits and sand dunes. Sand and gravel common in deeper buried channels.	GLACIAL DRIFT AQUIFER Sand and gravel, permeable. Some wells yield more than 1000 gpm. Specific capacities from 2.1 to 48 gpm/ft., average 13 gpm/ft. Coefficient of transmissibility from 3400 to 160,000 gpd/ft., average 33,000 gpd/ft.
PALEOZOIC	SILURIAN	NIAGARAN				Racine		300	Silty dolomite, some chert, porous reef zones; interreef zones argillaceous, silty and cherty.	SILURIAN AQUIFER Not consistent; some wells yield more than 1000 gpm. Crevices and solution channels more abundant near surface. Specific capacities from 0.1 to 500 gpm/ft. Highest average specific capacities (50.0 gpm/ft.) in Du Page County wells, lowest (3 gpm/ft.) in Lake County. Coefficient of transmissibility averages 140,000 gpd/ft. in Du Page County, 9000 gpd/ft. in Lake County.
						Waukesha		20-50	Silty dolomite, some chert, silt decreases toward base.	
						Joliet		20-75	Dolomite, cherty, pure near top, shale interbeds at base.	
		AL				Kankakee		20-50	Dolomite, with shale partings, some chert and glauconite.	
						Edgewood		0-100	Shaly dolomite and dolomitic shale, very cherty.	
	ORDOVICIAN	CINCINNATIAN	RICH		MAQUOKETA	Neda		0-15	Shale, with hematitic or limonitic oolites, some dolomite.	Shales, generally not water yielding, act as barriers between shallow and deep aquifers. Crevices in dolomite yield small amounts of water.
						Brainard		0-100	Silty dolomitic shale with silty dolomite interbeds.	
			MA			Ft. Atkinson		5-50	Fine to coarse-grained dolomite or limestone, some shale interbeds.	
			ED			Scales		90-100	Silty dolomitic shale, with interbeds of fine-grained, silty dolomite; phosphatic zone at base, and pyritic occasionally at top.	
		CHAMPLAINIAN	TRENTONIAN	OTTAWA	GALENA	Wise Lake-Dunleith		170-210	Fine to medium-grained dolomite.	TOP OF CAMBRIAN-ORDOVICIAN AQUIFER Where formation lies below shales, development and yields of crevices are small; where not capped by shales, dolomites are fairly permeable.
						Guttenberg		0-15	Fine to medium-grained dolomite, speckled, some shale partings.	
			BLACKRIVERAN		PLATTEVILLE	Nachusa		0-50	Fine to medium grained dolomite, cherty in upper part.	
						Grand Detour		20-40	Very fine-grained dolomite, mottled, some chert, shale partings, oolites.	
						Mifflin		20-50	Fine-grained dolomite with shale partings.	
						Pecatonica		20-50	Fine-grained dolomite, with thin argillaceous partings, sandy at base.	
					ANCELL	Glenwood		3-80	Dolomite, shale, and dolomitic sandstone.	Small to moderate quantities of water. Transmissibility probably about 15% of that of Cambrian-Ordovician aquifer.
						St. Peter		100-600	Fine to medium-grained sandstone, commonly unit of shale and chert rubble at base.	
		CANADIAN		KNOX	PRAIRIE DU CHIEN	Shakopee		0-67	Primarily fine-grained dolomite with beds of sandy dolomite, sandstone, and shale, oolitic chert common.	Crevices in dolomite and sandstone generally yield small amounts of water. Trempealeauan units locally well creviced and partly responsible for exceptionally high yields of several deep wells.
						New Richmond		0-35	Medium-grained partly dolomitic sandstone with oolitic chert.	
						Oneota		190-250	Medium to coarse-grained dolomite, cherty.	
						Gunter		0-15	Medium-grained, dolomitic sandstone with gray and green shale partings.	
	CAMBRIAN	CROIXAN	TREMPEALEAUAN			Eminence		50-150	Fine to medium-grained sandy dolomite with sandstone beds at or near base. Dolomite contains oolitic chert.	
						Potosi		90-220	Fine-grained dolomite, usually contains drusy quartz, some fine sand and glauconite in lower part, and some glauconite near top.	
			FRANCONIAN			Franconia		50-200	Fine-grained dolomitic sandstone in North, sandy dolomite to South, glauconitic, numerous red and green shale beds.	
						Ironton		80-130	Medium-grained, generally poorly sorted, dolomitic sandstone, beds of coarse-grained sandstone common.	Most productive units of Cambrian-Ordovician Aquifer; transmissibility probably about [illegible] of total. Coefficients of transmissibility and storage of the Cambrian-Ordovician Aquifer averages 17,000 gpd/ft. and 0.00035. BASE OF CAMBRIAN-ORDOVICIAN AQUIFER
						Galesville		10-100	Fine-grained, well sorted sandstone.	
			DRESBACHIAN			Eau Claire		370-575	Siltstone, shale, dolomite, and fine-grained sandstone, with some limestone locally. Glauconite abundant in some zones, pyrite incrusted sand near base. In southern part, more carbonates, clastics finer grained.	Shales, generally not water yielding, act as barrier between Ironton-Galesville and Mt. Simon.
				POTSDAM		Mt. Simon		[illegible]-2500	Medium-grained sandstone with some coarse-grained beds, especially in upper part. Some zones of quartz pebbles and beds of shale.	MT. SIMON AQUIFER Moderate amounts of water; permeability intermediate between that of Glenwood - St. Peter and Ironton - Galesville.
PRECAMBRIAN										

Geologic data for Cambrian and Ordovician adapted from Illinois State Geological Survey Report of Investigations 218 by Mr. T.C. Buschbach, 1964. Data for Silurian from Illinois State Geological Survey Reprint [illegible], The Silurian strata of Northeastern Illinois, by Mr. H.B. Willman, 1962.

* Data adapted from Preliminary Report on Ground-Water Resources of the Chicago Region, Illinois. Cooperative Ground-Water Report 1, Illinois State Water and Geological Surveys, 1959.

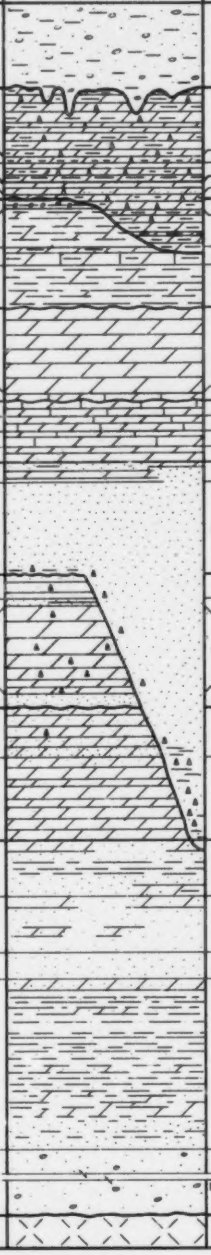


CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

GENERALIZED COLUMN

HARZA ENGINEERING COMPANY DWG. NO. 397 P22
BAUER ENGINEERING, INC. PROJECT 6524

GENERALIZED GEOLOGIC COLUMN

ERA	SYSTEM	SERIES	STAGE	MEGA-GROUP	GROUP	FORMATION	GRAPHIC COLUMN	THICKNESS (feet)	LITHOLOGY	WATER-YIE
CENOZOIC	QUATERNARY	PLEISTOCENE						0-400	Consists primarily of tills and lacustrine clays, with some stratified outwash deposits and sand dunes. Sand and gravel common in deeper buried channels.	Sand and gravel, pe-meable capacities from 2.1 tc 56 gp transmissibility from 3400
PALEOZOIC	SILURIAN	NIAGARAN				Racine		300	Silty dolomite, some chert, porous reef zones; interreef zones argillaceous, silty and cherty.	Not consistent; some wells channels more abundant near 550 gpm/ft. Highest avera County wells, lowest (5 gp missibility averages 100,0 Lake County.
						Waukesha		20-30	Silty dolomite, some chert, silt decreases toward base.	
						Joliet		50-75	Dolomite, cherty, pure near top, shale interbeds at base.	
		AL.				Kankakee		20-50	Dolomite, with shale partings, some chert and glauconite.	
						Edgewood		0-100	Shaly dolomite and dolomitic shale, very cherty.	
	ORDOVICIAN	CINCINNATIAN	RICH.		MAQUOKETA	Neda		0-15	Shale, with hematitic or limonitic oolites, some dolomite.	Shales, generally not water deep aquifers. Crevices i
						Brainard		0-100	Silty dolomitic shale with silty dolomite interbeds.	
						Ft. Atkinson		5-50	Fine to coarse-grained dolomite or limestone, some shale interbeds.	
			MA. ED.			Scales		90-100	Silty dolomitic shale, with interbeds of fine-grained, silty dolomite; phosphatic zone at base, and occasionally at top.	
		CHAMPLAINIAN	TRENTONIAN	OTTAWA	GALENA	Wise Lake - Dunleith		170-210	Fine to medium-grained dolomite.	TOP OF Where formation lies below small; where not capped by
						Guttenberg		0-15	Fine to medium-grained dolomite, speckled, some shale partings.	
			BLACKRIVERAN		PLATTEVILLE	Nachusa		0-50	Fine to medium-grained dolomite, cherty in upper part.	
						Grand Detour		20-40	Very fine-grained dolomite, mottled, some chert, shale partings, oolites.	
						Mifflin		20-50	Fine-grained dolomite with shale partings.	
						Pecatonica		20-50	Fine-grained dolomite, with thin argillaceous partings, sandy at base.	
					ANCELL	Glenwood		0-80	Dolomite, shale, and dolomitic sandstone.	Small to moderate quantiti 15% of that of Cambrian-Or
						St. Peter		100-600	Fine to medium-grained sandstone, commonly unit of shale and chert rubble at base.	
		CANADIAN		KNOX	PRAIRIE DU CHIEN	Shakopee		0-67	Primarily fine-grained dolomite with beds of sandy dolomite, sandstone, and shale, oolitic chert common.	Crevices in dolomite and Trempealeauan units locally exceptionally high yields
						New Richmond		0-35	Medium-grained partly dolomitic sandstone with oolitic chert.	
						Oneota		190-250	Medium to coarse-grained dolomite, cherty.	
						Gunter		0-15	Medium-grained, dolomitic sandstone with gray and green shale partings.	
	CAMBRIAN	CROIXAN	TREMPLEAUAN			Eminence		50-150	Fine to medium-grained sandy dolomite with sandstone beds at or near base. Dolomite contains oolitic chert.	
						Potosi		90-220	Fine-grained dolomite, usually contains drusty quartz, some fine sand and glauconite in lower part, and some glauconite near top.	
			FRANCONIAN			Franconia		50-200	Fine-grained dolomitic sandstone in North, sandy dolomite to South, glauconitic, numerous red and green shale beds.	
						Ironton		80-130	Medium-grained, generally poorly sorted, dolomitic sandstone, beds of coarse-grained sandstone common.	Most productive units of C probably about 80% its' t storage of the Cambrian-Or 0.00035. BASE OF
			DRESBACHIAN			Galesville		10-100	Fine-grained, well sorted sandstone.	
						Eau Claire		370-575	Siltstone, shale, dolomite, and fine-grained sandstone, with some limestone locally. Glauconite abundant in some zones, pyrite incrusted sand near base. In southern part, more carbonates, clastics finer grained.	Shales, generally not water Galesville and Mt. Simon.
				POTSDAM		Mt. Simon		1200-2900	Medium-grained sandstone with some coarse-grained beds, especially in upper part. Some zones of quartz pebbles and beds of shale.	Moderate amounts of water Glenwood - St. Peter and
PRECAMBRIAN										

Geologic data for Cambrian and Ordovician adapted from Illinois State Geological Survey Report of Investigations #218 by Mr. T.C. Buschbach, 1964. Data for Silurian from Illinois State Geological Survey Reprint #1962 M, The Silurian strata of Northeastern Illinois, by Mr. H.B. Willman, 1962.

* Data adapted from Pre
Chicago Region, Illino
State Water and Geologi

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BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT B-3

LOGIC COLUMN

OGY	WATER-YIELDING PROPERTIES *
ine clays, with some stratified and gravel common in deeper	**GLACIAL DRIFT AQUIFER** Sand and gravel, permeable. Some wells yield more than 1000 gpm. Specific capacities from 2.1 to 68 gpm/ft., average 12 gpm/ft. Coefficient of transmissibility from 3000 to 100,000 gpd/ft., average 25,000 gpd/ft.
f zones; interreef zones argillaceous,	**SILURIAN AQUIFER** Not consistent; some wells yield more than 1000 gpm. Crevices and solution channels more abundant near surface. Specific capacities from 0.1 to 550 gpm/ft. Highest average specific capacities (54.4 gpm/ft.) in Du Page County wells, lowest (5 gpm/ft.) in Lake County. Coefficient of transmissibility averages 100,000 gpd/ft. in Du Page County, 9000 gpd/ft. in Lake County.
es toward base.	
interbeds at base.	
rt and glauconite.	
herty.	
des, some dolomite.	Shales, generally not water yielding, act as barriers between shallow and deep aquifers. Crevices in dolomite yield small amounts of water.
ite interbeds.	
stone, some shale interbeds.	
fine-grained, silty dolomite; ty at top.	
	TOP OF CAMBRIAN-ORDOVICIAN AQUIFER Where formation lies below shales, development and yields of crevices are small; where not capped by shales, dolomites are fairly permeable.
ed, some shale partings.	
in upper part.	
chert, shale partings, oolites.	
ce.	
aceous partings, sandy at base.	
e.	
aly unit of shale and chert rubble	Small to moderate quantities of water. Transmissibility probably about 15% of that of Cambrian-Ordovician aquifer.
s of sandy dolomite, sandstone, and	Crevices in dolomite and sandstone generally yield small amounts of water. Trempealeauan units locally well creviced and partly responsible for exceptionally high yields of several deep wells.
with oolitic chert.	
ty.	
h gray and green shale partings.	
with sandstone beds at or near base.	
drusty quartz, some fine sand and conite near top.	
th, sandy dolomite to South, le beds.	
, dolomitic sandstone, beds of	Most productive units of Cambrian-Ordovician Aquifer; transmissibility probably about 80% its' total. Coefficients of transmissibility and storage of the Cambrian-Ordovician Aquifer averages 17,400 gpd/ft. and 0.00035. **BASE OF CAMBRIAN-ORDOVICIAN AQUIFER**
rained sandstone, with some limestone zones, pyrite incrusted sand near base. tics finer grained.	Shales, generally not water yielding, act as barrier between Ironton - Galesville and Mt. Simon.
	MT. SIMON AQUIFER
rse-grained beds, especially in upper d bed- of shale.	Moderate amounts of water; permeability intermediate between that of Glenwood - St. Peter and Ironton - Galesville.

* Data adapted from Preliminary Report on Ground-Water Resources of the Chicago Region, Illinois. Cooperative Ground-Water Report 1, Illinois State Water and Geological Surveys, 1959.

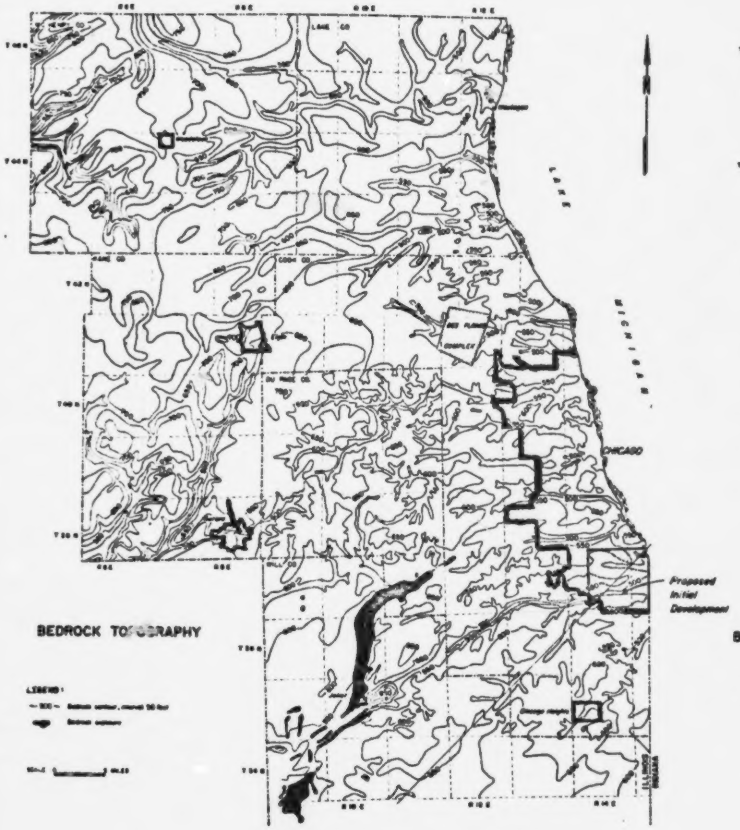
LEGEND

- *Glacial drift*
- *Limestone*
- *Dolomite*
- *Shaly dolomite*
- *Shale*
- *Sandstone*
- *Dolomitic sandstone*
- *Conglomeratic sandstone*
- ▲ △ *Chert*
- ▲ ▲ *Oölitic chert*
- *Oölites*
- *Crystalline complex*

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

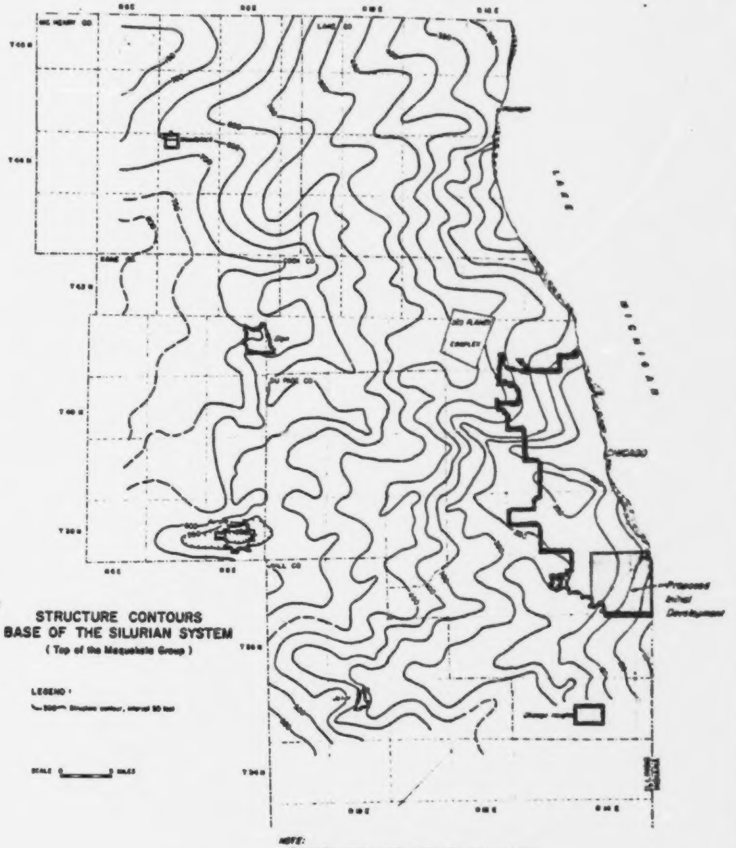
GENERALIZED COLUMN

EXHIBIT B-4


LAKE MICHIGAN
CHICAGO
LAKE CO
COOK CO
DU PAGE CO
WILL CO
KANE CO
DES PLAINES COMPLEX
ILLINOIS
INDIANA
Proposed Initial Development
LEGEND:
— 500 — Bedrock contour, interval 50 feet
Bedrock outcrops
SCALE 0 5 MILES

BEDROCK TOPOGRAPHY

NOTE:
Contours from current work sheets, Illinois State Geological Survey, Naperville Office, December, 1962.


LAKE MICHIGAN
CHICAGO
McHENRY CO
LAKE CO
KANE CO
COOK CO
DU PAGE CO
WILL CO
DES PLAINES COMPLEX
ILLINOIS
INDIANA
Proposed Initial Development
LEGEND:
— 300 — Structure contour, interval 50 feet
SCALE 0 5 MILES

STRUCTURE CONTOURS
BASE OF THE SILURIAN SYSTEM
(Top of the Maquoketa Group)

NOTE:
Contours from current work sheets, Illinois State Geological Survey, Naperville Office, December, 1962.

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

SILURIAN CONTOURS

HARZA ENGINEERING COMPANY DWG NO 387 P 25
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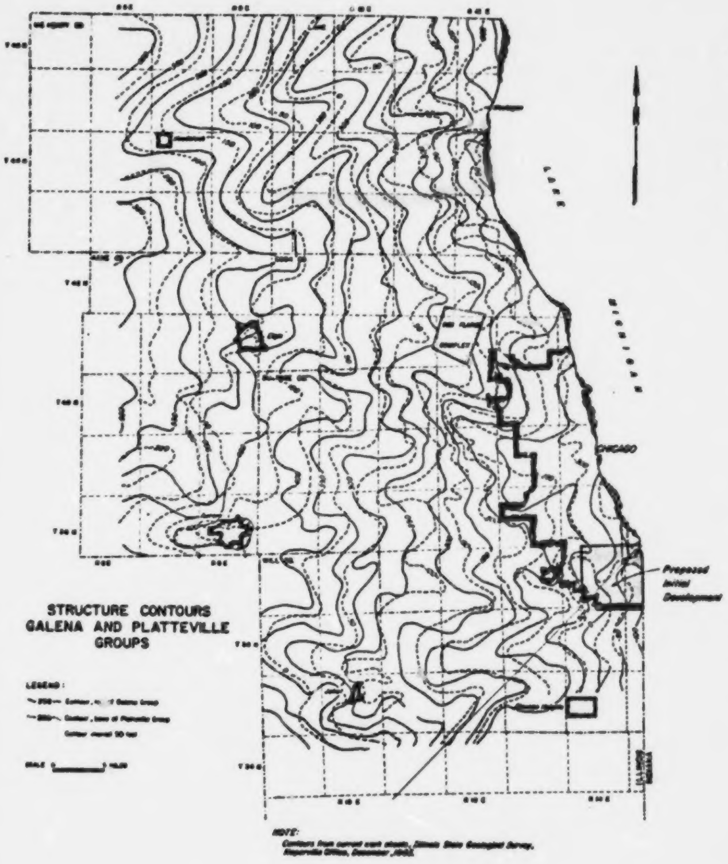
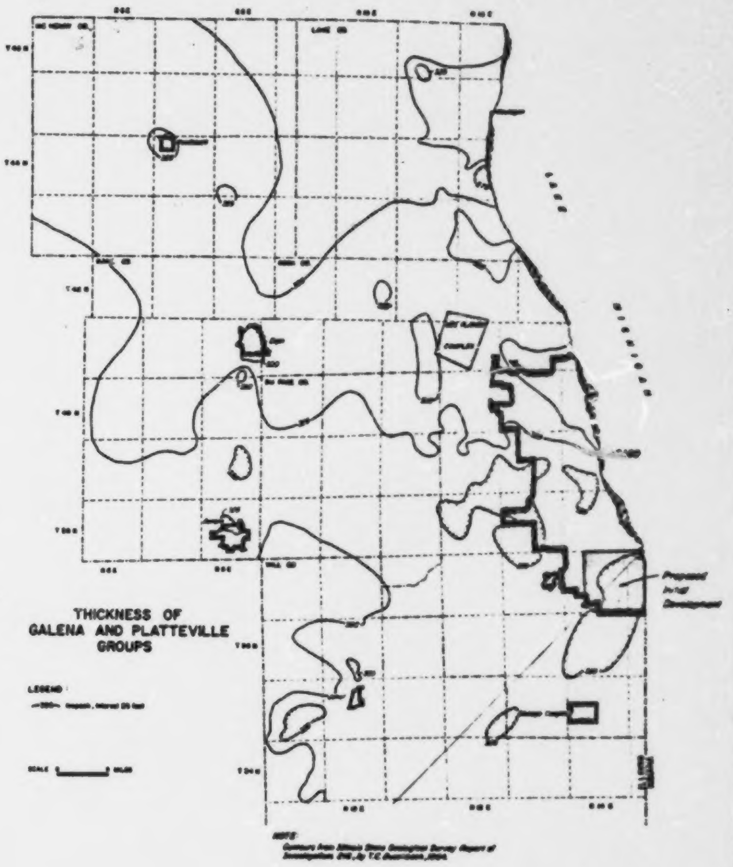
EXHIBIT B-5


CHICAGO
Proposed Initial Development
LEGEND:
NOTE:

STRUCTURE CONTOURS
GALENA AND PLATTEVILLE
GROUPS


Proposed Initial Development
LEGEND:
NOTE:

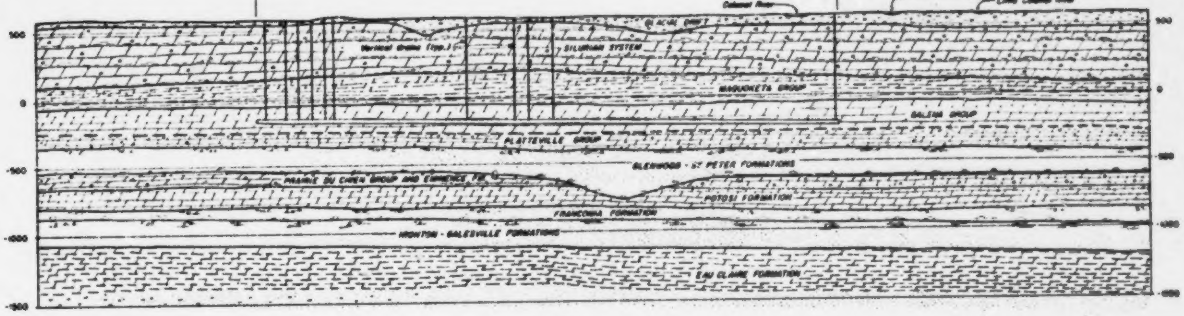
THICKNESS OF
GALENA AND PLATTEVILLE
GROUPS

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

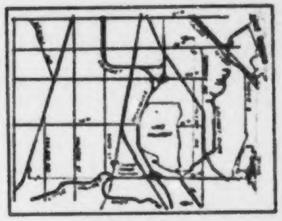
GALENA-PLATTEVILLE CONTOURS

HARZA ENGINEERING COMPANY DWG NO 387 P26
BAUER ENGINEERING, INC. PROJECT 6524

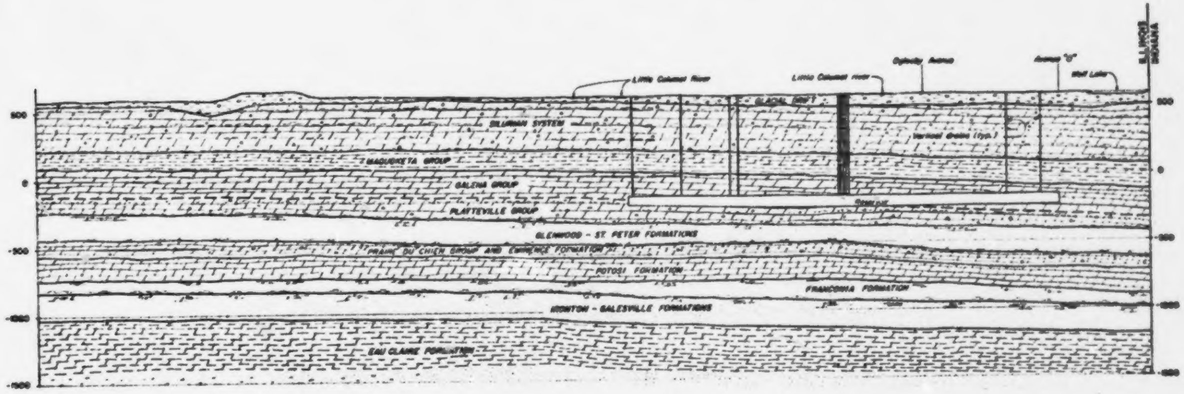
EXHIBIT B-6


67th Street
130th Street
Calumet River
Little Calumet River
GLACIAL DRIFT
Vertical Drains (typ.)
SILURIAN SYSTEM
MAQUOKETA GROUP
GALENA GROUP
PLATTEVILLE GROUP
GLENWOOD - ST PETER FORMATIONS
PRAIRIE DU CHIEN GROUP AND EMINENCE FM.
POTOSI FORMATION
FRANCONIA FORMATION
IRONTON - GALESVILLE FORMATIONS
EAU CLAIRE FORMATION
500
0
-500
-1000
-1500

SECTION ALONG OGLESBY AVENUE



KEY PLAN


Little Calumet River
Little Calumet River
Oglesby Avenue
Avenue "O"
Wolf Lake
ILLINOIS
INDIANA
GLACIAL DRIFT
SILURIAN SYSTEM
Vertical Drains (typ.)
MAQUOKETA GROUP
GALENA GROUP
PLATTEVILLE GROUP
GLENWOOD - ST PETER FORMATIONS
PRAIRIE DU CHIEN GROUP AND EMINENCE FORMATION
POTOSI FORMATION
FRANCONIA FORMATION
IRONTON - GALESVILLE FORMATIONS
EAU CLAIRE FORMATION
500
0
-500
-1000
-1500

SECTION ALONG 130th STREET

NOTES:

1. [illegible]
2. For descriptions of geologic units and symbols used on sections, see Exhibit B-5, Generalized Geologic Column.
3. Vertical exaggeration of sections approximately 7x.

Scale 0 1 Mile
Horizontal Scale

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES
GEOLOGIC SECTIONS

HARZA ENGINEERING COMPANY DWG. NO. 387 P 24
BAUER ENGINEERING, INC. PROJECT 6524

Appendix C

WATER QUALITY CONSIDERATIONS

Introduction

This appendix describes (a) how the proposed Deep Tunnel Plan would reduce the pollution load reaching Illinois streams, (b) how it would reduce hazards to public health and safety which are caused by local sewer back-up and flooding, and (c) how certain elements might be designed from a sanitary engineering viewpoint. It also considers the possible use of the Deep Tunnel Plan as a sewage treatment system. The discussion is organized as follows:

(a) Water Quality Problems in the Proposed Plan.
    (1) Neighborhood Drainage and Sewer Systems
    (2) Vertical Drains
    (3) BOD Load
    (4) Capacity of Existing Treatment Plant
    (5) Sediment Deposition
    (6) Ventilation
    (7) Hydroelectric Plant

(b) The Proposed Plan as a Sewage Treatment System
    (1) Inherent Treatment Processes
    (2) Enhanced Treatment Processes

Water Quality Problems in the Proposed Plan

The recommended Deep Tunnel Plan is described in report Chapter III; the proposed arrangement is shown in report Exhibit 7; and principal characteristics are summarized in Table C-1. The performance of the

TABLE C-1

RECOMMENDED DEEP TUNNEL PLAN

Total drainage area	13,500 acres (21.0 sq. mi.)
Enlarged sewer system and vertical drain discharge capacity	27,000 c.f.s. (2.0 inches/hour)
Underground reservoir volume	2820 acre-feet (2.5 inches runoff)
Pumping capacity	3750 c.f.s. (0.25 inches/hour)
Generating capacity	300,000 kilowatts
Surface reservoir volume	2360 acre-feet (2.1 inches runoff)
Power water volume	1.0 inch runoff (1120 acre-feet)
Length of additional sewers	29.8 miles
Length of tunnels	23.4 miles
Number of vertical drains	68
Area of surface reservoir	145 acres of land

various elements of the Plan from a health and sanitary engineering viewpoint is described in the following sections.

Neighborhood Drainage and Sewer System

The elimination of temporary storage of contaminated water in basements reduces hazards to health. In Appendix A, the frequency and amounts of temporary storage in basements were analyzed, with and without the proposed Deep Tunnel Plan. Nearly all basement flooding and resultant hazards would be eliminated by the proposed system.

Lateral sewers and main sewers.tend to be flushed out during storm flows by the higher velocities induced by the vertical drains. This flushing should reduce routine maintenance costs for the lateral systems.

Vertical Drains

The vertical drains would act as spillways to convey the stormwater overflows about 800 feet down from combined sewers to plunge pools adjacent to the tunnels comprising the lower reservoir. A great deal of air would be entrained, and solids would generally be reduced in size by this process. It is likely that the polluted water would be fully saturated with dissolved exygen as it would enter the tunnels. Trash passed down the shafts could be sizable, as the diameters of some of the vertical drains in the recommended Deep Tunnel Plan would be several feet.

BOD Load

The frequency of overflows developed in Appendix A must be related to quality of the water measured in terms of five-day BOD and other conventional parameters. Stormwater quality data were furnished by the

Public Health Service[1] from their study of the Roscoe Street sewer, which covers a period of about one year of flow records for 8.6 square miles of urban Chicago. That study gave quantitative data on how quality of overflows may be expected to vary with duration of overflow.

The results are summarized in Exhibit C-1. Examination of these data reveals the following facts:

(a) Overflows of more than four hours duration have five-day BOD's of about 20 to 25 mg/l. This is about the same as the effluent from a trickling-filter sewage treatment plant.

(b) Volumes of overflow tend to increase with the duration of overflow. There is considerable scatter in this relationship, which is evident when storms are plotted individually. However, a general trend line of 0.04 inches of overflow per hour of overflow duration is a reasonable approximation of the data.

(c) Volumes of overflow tend to increase.with volumes of total runoff. A trend line of 0.83 ratio (overflow/runoff) is a good approximation.

(d) The rainfall-runoff relationship is not well defined, but appears to vary considerably with season and intensity of rainfall. Some of the scatter is undoubtedly due to inaccurate rainfall data; nevertheless, useful relationships can be derived. A simple one is to assume that the average ratio of runoff/rainfall equals 0.40. It is interesting to note that this ratio is nearly the same as the portion of impervious area to the total area (42.5 percent). Of the 34 storms analyzed, 30 have runoff/rainfall ratios falling between 0.19 and 0.66, the average being 0.40.

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[1] Department of Health, Education and Welfare. Federal Water Pollution Control Administration, Region V. Great Lakes and Illinois River Basins Project. Chicago.

Exhibt C-2 is developed from Exhibit C-1, the use of a 13,500-acre drainage area of the recommended Deep Tunnel Plan, and the array of storms for the average year as presented in Appendix A. Exhibit C-2 may be summarized as follows:

		21.0 Square Mile Area	
Storms equal to or less than the following depth of rainfall	Number of Events per year	Pounds of 5-day BOD to be expected per year in lower reservoir (present land use)	Pounds of 5-day BOD to be expected per year in lower reservoir (ultimate land use)
0.2 inches	78	287,000 lbs	413,000 lbs
0.6 inches	107	560,000 lbs	813,000 lbs
1.0 inches	115	653,000 lbs	947,000 lbs
2.0 inches	121	747,000 lbs	1,087,000 lbs
6 inches	122	793,000 lbs	1,133,000 lbs

Another approach to the calculation of BOD of overflows to the lower reservoir is shown on Echibits C-3 and C-4. It is based on the assumption that the quality of storm-water overflows is the same as quality of influent to the Calumet Sewage Treatment Works during and after rainfalls. The points plotted on Exhibit C-3 are taken from the past 10 years of daily records at this plant. On this exhibit is plotted the curve derived from the Roscoe Street Sewer data (Exhibit C-1), as well as curves labeled "Selected Value for Plotting Exhibit C-4" These latter curves are conservative approximations of the relationship between rainfall and quality of sewage, as most of the plotted points lie below and to the left of these curves.

Exhibit C-4 summarizes the BOD and settleable solids delivered to the lower reservoir from the 13,500 acre watershed on the basis of the assumptions of Exhibit C-3. (Numerical values on Exhibits C-2 and C-4 relate to a drainage area of 19,000 acres and were adjusted by a ratio of 0.71.) The total BOD load can be summarized as follows:

	21.0 Square Mile Area	
Storms equal to or less than the following depth of rainfall	Pounds of 5-day BOD per year delivered to lower reservoir (present land use)	Pounds of 5-day BOD per year delivered to lower reservoir (ultimate land use)
0.2"	427,000 lbs	607,000 lbs
0.6"	940,000 lbs	1,287,000 lbs
1.0"	1,153,000 lbs	1,620,000 lbs
2.0"	1,447,000 lbs	2,067,000 lbs
6.0"	1,776,000 lbs	2,256,000 lbs

Capacity of Existing Treatment Plant

It is interesting to note that rainfalls of 2 inches or less are responsible for nearly all of the total BOD which would enter the tunnel system. These storms would have runoffs of 0.7 inches or less, amounting at most to 800 acre-feet, or 270 million gallons. The capacity of the Calumet Sewage Treatment Works is as follows:

Maximum capacity	330 mgd
Dry weather flow (present)	150 mgd
Dry weather flow (future)	220 mgd
Storm-water treatment capacity (future)	110 mgd

Spreading of the 270 million gallons of storm-water overflow over three days would permit treatment in the present Calumet Works, assuming the 110 mgd of future storm water treatment capacity would be available for this purpose. The pumping capacity required of about 150 cfs to deliver 270 million gallons from the lower reservoir is a small fraction of the capability of the pumped-storage power station.

It is doubtful, however, that it would be necessary to treat storm-water overflows in the Calumet Sewage Treatment Works. The reasons are developed in the subsequent section on the use of the Deep Tunnel Plan as a sewage-treatment system.

Sediment Deposition

The total amount of settleable solids* is estimated with the aid of exhibits C-2, C-3, and C-4 as follows;

Item	Estimate from Roscoe Street Sewer Overflow Data	Estimate from Calumet Treatment Works Influent Data
Total solids, tons per year present land use	730 tons	1,465 tons
Total solids, tons per year ultimate land use	1,063 tons	2,085 tons
Solids transported to surface reservoir through natural entrainment in daily power generation flows (20% assumed) ultimate land use	210 tons	415 tons
Remaining solids, ultimate land use, total in tons per year	853 tons	1,670 tons
Grit (dry)	426 tons	835 tons
Sludge (dry)	426 tons	835 tons

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* Settleable solids are reported as milligrams of dry solids per litre of mixture in the Roscoe Street Sewer data. Data from the Calumet Sewage Treatment Works are reported as suspended solids, in milligrams per liter of sample (obtained by filtration). It was assumed that one-half of the suspended solids would be settleable solids.

Item	Estimate from Roscoe Street Sewer Overflow Data	Estimate from Calumet Treatment Works Influent Data
Remaining solids, ultimate land use, total in cubic yards per year	5,427 cu. yd.	10,570 cu. yd.
Grit (wet)	293 cu. yd.	570 cu. yd.
Sludge at 10% solids	5,134 cu. yd.	10,000 cu. yd.

The highest figure of 10,570 cubic yards per year is 0.23 percent of the proposed underground reservoir volume of 4.5 million cubic yards. Thus, it would require about 40 years to develop an accumulation of 10 percent of the underground reservoir volume. Annual removal of 10,000 cubic yards per year could be accomplished during annual maintenance periods which would be carried out during cold weather. Using 10 trucks of 10 cubic yeards each and making 1/2-hour round trips for average hauls of three miles, the annual deposition could be removed in about four working days.

Ventilation

A mechanical ventilation system would be required for two purposes:

(a) To insure the presence of an adequate amount of oxygen in the underground reservoir and to prevent accumulation of undesirable gases.

(b) To evacuate air from the underground reservoir at the same rate as it would be displaced by the inflowing water and entrained air.

The first pupose would require a relatively small ventilation capacity, probably on the order of 200,000 cubic feet per minute. The second purpose would require a capacity greater than the aggregate hydraulic capacity of all vertical drains. It was assumed for the purpose of these studies that the ventilation capacity would be twice the aggregate hydraulic inflow capacity or 3,000,000 cubic feet per minute. further studies of the hydralics of vertical drains and of the movement of air in the underground reservoir would be carried our during feasibility and engineering design studies.

For the purposes of estimating the cost of a mechanical ventilation system, it was assumed that all of the air evacuated from the underground reservoir during the design storm would be passed through large fans and carried to the atmosphere through a vertical shaft surmounted by a large chimney.

Hydroelectric Plant Operation

Most of the trash carried by the auxiliary relief sewers would be carried into the lower reservoir. Because it would be passed through the violent turbulence in the plunge pool, debris would probably be broken up into relatively small pieces. A small amount of trash could be expected to accumulate on the trash racks over the draft-tube outlets of the hydroelectric station. This could be ground up and added to the water, so that the end result would be a deposition of trash either on the bottom of the tunnels or in the surface reservoir.

The expected range of pH values is not so extreme as to require any special provision beyond that normally used for sewage handling equipment to guard against corrosion. The large volume of water stored in the system would tend to minimize short-term fluctuations in water quality.

The Proposed Plan as a Sewage Treatment System

Inherent Treatment Process

Exhibit C-5 shows the method in which the Deep Tunnel Reservoir Plan could function as a treatment process. The functioning would be as follows:

(a) Air and waste water would enter the system through the vertical drains to drop eight hundred feet into the plunge pools. Repeated impact during the fall and intense turbulence in the plunge pools would tend to reduce the size of solids. More surface area would thereby be exposed to bacteriological decomposition.

(b) A settling of solids would take place, with the heavier materials being deposited for subsequent removal by excavation equipment. The daily back-and-forth movement of water in the lower reservoir would stir up and entrain the lighter solids and lead to their partial decomposition in the surface reservoir.

(c) Air drawn into the system and mechanically exhausted would maintain conditions in the underground reservoir favorable to oxygen transfer.

(d) The surface reservoir, 135 acres in area, would function as an oxidation pond, possibly with baffling and mechanical aeration to enhance performance.

A simple computation will demonstrate the possiblities of this procedure. Assume a maximum total load from storm-water overflows of 2,256,000 pounds of BOD per year to be spread over 365 days. This would give an average load of 6200 pounds per day, corresponding to 46 pounds of five-day BOD per acre of the 135-acre surface reservoir. (In calculating benefits a lesser figure of 4,500 lb. per day was used based upon a proration of U. S. P. H. S. estimate for the Chicago metropolitan area.) Such a loading is high for a relatively quiescent oxidation pond to give good results. However, when it is considered that many of the inherent aspects of operation of the Deep Tunnel Plan would enhance aeration processes, it appears quite possible that the system would be an adequate treatment process for the polluted storm water which would enter it, without further treatment in the Calumet Sewer Treatment Works.

Some of the favorable aspects of the Deep Tunnel Plan which contribute to the treatment process are:

(a) The initial drop into the plunge pools,

(b) The complete lower reservoir ventilation,

(c) The back-and-forth movement of water in the lower reservoir,

(d) The currents set up in the surface reservoir by discharges into and out of the penstocks, and

(e) The nearly constant temperature provided by the heat exchange with the subterranean rocks.

Enhanced Treatment Process

There may be other ways in which the Deep Tunnel Plan could be developed as a sewage treatment process. Some potential is apparent for the enhanced system to provide for treatment of dry-weather flows as well as for storm water. More complete investigations of this aspect of the Plan must await the feasibility studies which are proposed to follow. At this stage, however, a few observations are possible.

The fundamental idea of using power to oxygenate waste water is well accepted. Many different devices have been used, among them turbines, pumps, jets, and high-pressure blowers. The amount of oxygen absorbed into the water varys from 20 to 80 pounds per horsepower-day. The amount of oxygen absorbed ranges from 10 to 20 percent of the amount supplied.

The Plan includes a substantial power installation. For example, the recommended Plan (1C) would have 300 MW of generating capacity. A two-foot loss in head on the turbines resulting from bleeding in air at the throat of the draft tube would reduce the power output by 0. 3 percent or 1200 horse-power. Applying this amount of power and assuming 20 pounds of oxygen added per horsepower-day, 24, 000 pounds of oxygen could be added during the day. (Air also could be added to the suction side of the pumps throughout the pumping cycle as is practised at the Yards Creek pumped-storage station in New Jersey.) This 24, 000 pounds would be more than sufficient to treat the average 6200 pounds of storm-water BOD, plus the 16, 000 pounds required to reduce the BOD of the effluent from the Calumet Sewage Treatment Works from 15 to 5 mg/l.

Preliminary computations indicate that about 4 ppm of dissolved oxygen could be added per pass through the turbines, or 8 ppm per day. On this basis, hypothetical computations were made assuming passage of all of the effluent of the Calumet Sewage Treatment Works through the system with provision for a one-day holding time in the surface reservoir. It was further assumed that a common storm occurred on a particular day, bringing additional BOD into the underground reservoir. The mixture of water would be made up as follows,

Source of Water	Millions of pounds of water	5-day BOD mg/l	5-day BOD pounds
Power water in storage	3070	5	15,300
Daily C.S.T.W. effluent	1600	15	24,000
Small storm	110	71	7,800
Totals	4780		47,100

Assuming two passes per day through the turbines, with oxygen being added during each pass, it appears that only a fraction of the assumed 4 ppm increment per pass would be required to treat the mixture in one day;

Half-day	5-day BOD ppm		Dissolved Oxygen, ppm			
Interval	Initial	Final	Initial	Loss	Gain	Final
1	9.9	6.9	10.0	3.0	2.0	9.0
2	6.9	4.9	9.0	2.0	2.0	9.0

Water would be released at 4.9 ppm BOD and 9.0 ppm dissolved oxygen. There is obviously still spare treatment capacity to be utilized under these assumed conditions.

Other preliminary computations indicate that treatment of a major storm would be accomplished in three days with an effluent of less than 5 mg/l BOD and 10 ppm DO if:

(a) The surface reservoir would be enlarged to contain all of the storm runoff less the power water which could be released to the river as the storm water was being pumped into the surface reservoir.

(b) The discharge of effluent from the Calumet Sewage Treatment Works into the underground reservoir would be suspended for the three-day period.

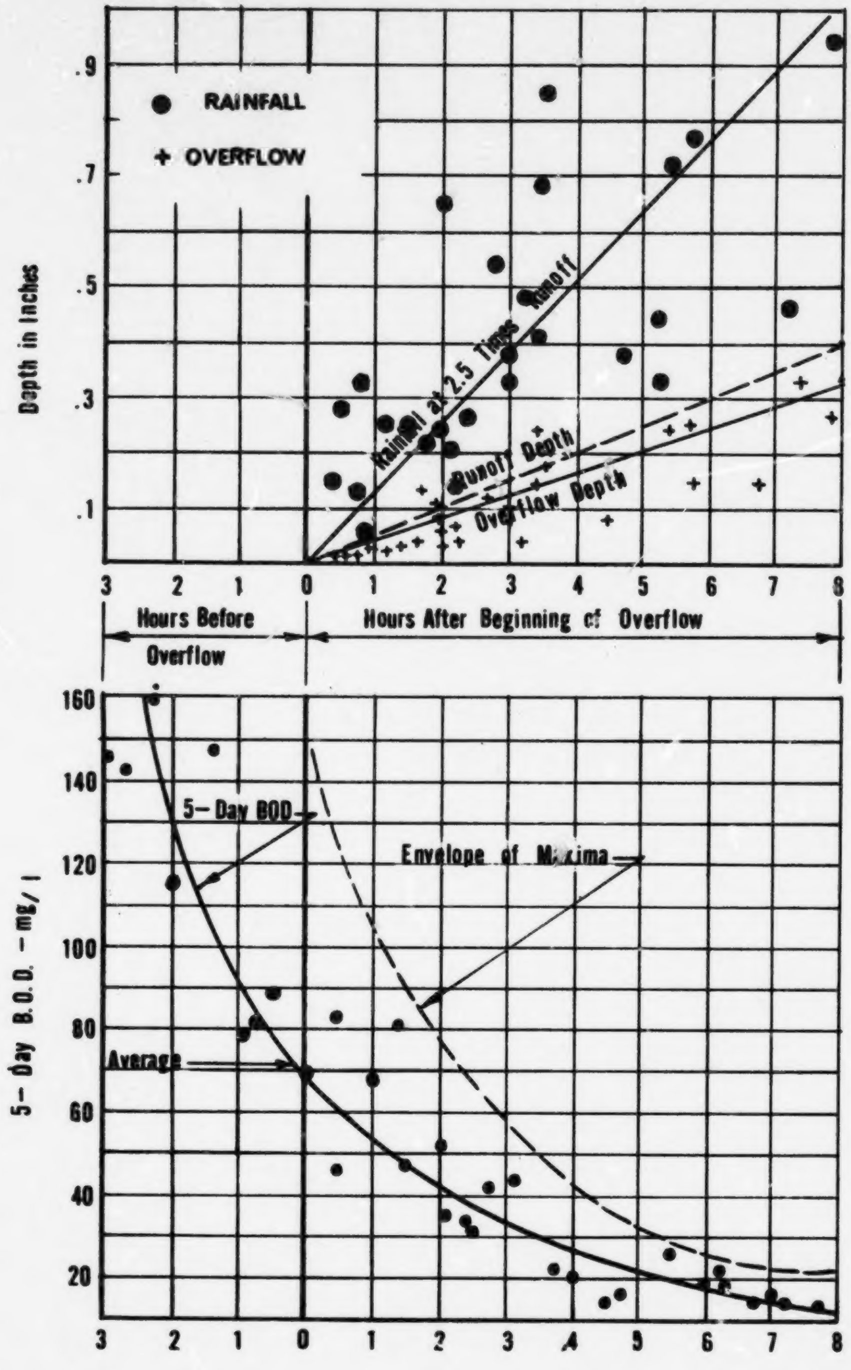
Other preliminary computations indicated the possibility of introducing a limited amount of dry-weather raw sewage into the underground reservoir to take advantage of the total potential treatment capacity of the proposed system.

The total potential treatment capacity with a 300,000 kilowatt installation is presently estimated to be on the order of 20,000 to 30,000 pounds of BOD per day. Further studies are required to explore the potential, which will depend on the amount of power that could be devoted to this purpose and to the physical facilities provided for it.

The potential value of the system as a sewage-treatment process is also to be investigated more thoroughly in the feasibility studies. It is sufficient to point out at this stage that there is a very great potential.

Acknowledgement

Dr. Don E. Bloodgood and Dr. G. H. Toebes of Purdue University acted as consultants during the sanitary engineering investigations, reviewed preliminary drafts of this appendix, and made valuable comments, criticisms, and suggestions. Their assistance and encouragement is gratefully acknowledged.



NOTE:

Data adapted from U.S. Public Health Service, Roscoe St. sewer studies.

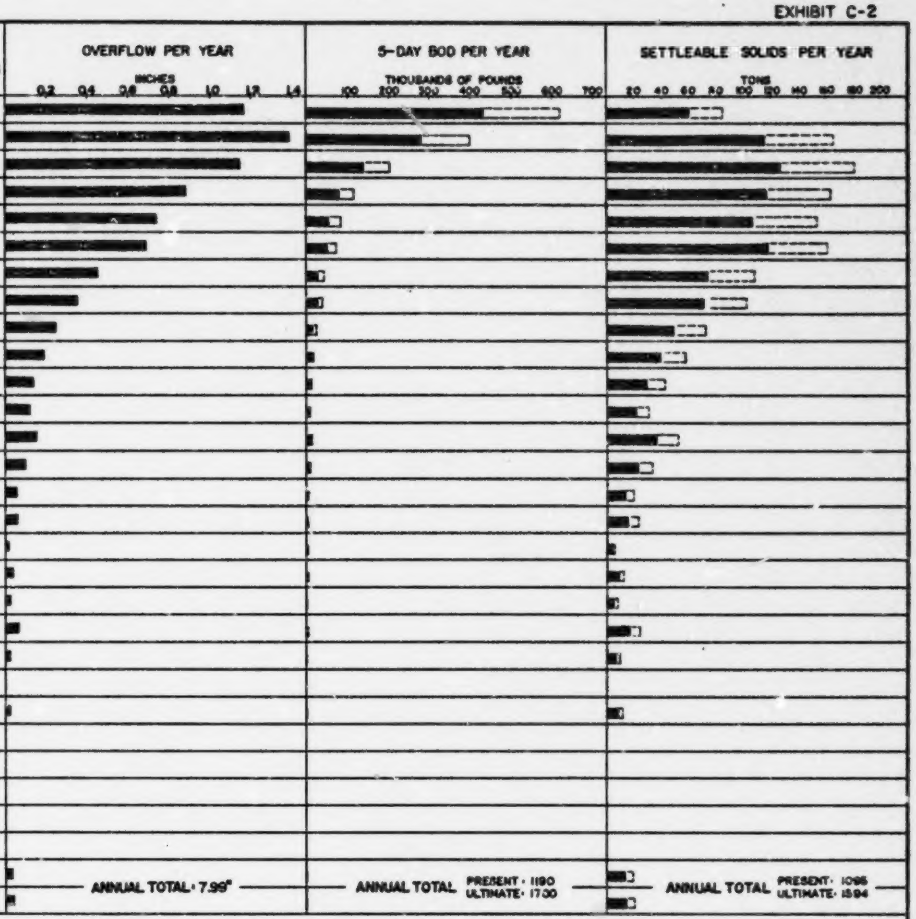
CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

QUALITY OF COMBINED SEWER OVERFLOWS

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT C-2

RAINFALL INTERVAL INCHES	AVERAGE RAINFALL INCHES	AVERAGE RUNOFF INCHES	AVERAGE OVERFLOW INCHES	AVERAGE 5-DAY BOD MG/L	AVERAGE SETTLEABLE SOLIDS MG/L	NUMBER OF EVENTS
0.0 TO 0.2	0.1	0.03	0.025	120	33	78
0.2 TO 0.4	0.3	0.09	0.07	65	54	18.6
0.4 TO 0.6	0.5	0.15	0.12	40	71	9.8
0.6 TO 0.8	0.7	0.21	0.17	30	84	5.2
0.8 TO 1.0	0.9	0.28	0.23	25	95	3.2
1.0 TO 1.2	1.1	0.35	0.29	24	108	2.4
1.2 TO 1.4	1.3	0.42	0.34	23	114	1.3
1.4 TO 1.6	1.5	0.49	0.41	23	122	0.92
1.6 TO 1.8	1.7	0.56	0.47	23	130	0.53
1.8 TO 2.0	1.9	0.63	0.53	23	138	0.36
2.0 TO 2.2	2.1	0.70	0.58	23	145	0.22
2.2 TO 2.4	2.3	0.76	0.64	23	152	0.14
2.4 TO 2.6	2.5	0.87	0.72	23	158	0.21
2.6 TO 2.8	2.7	1.00	0.83	23	164	0.106
2.8 TO 3.0	2.9	1.18	0.98	23	170	0.053
3.0 TO 3.2	3.1	1.35	1.12	23	176	0.053
3.2 TO 3.4	3.3	1.56	1.29	23	182	0.011
3.4 TO 3.6	3.5	1.75	1.46	23	187	0.021
3.6 TO 3.8	3.7	1.95	1.63	23	192	0.011
3.8 TO 4.0	3.9	2.15	1.79	23	197	0.032
4.0 TO 4.2	4.1	2.35	1.96	23	202	0.011
4.2 TO 4.4	4.3	2.55	2.13	23	207	—
4.4 TO 4.6	4.5	2.75	2.29	23	212	0.011
4.6 TO 4.8	4.7	2.95	2.46	23	217	—
4.8 TO 5.0	4.9	3.15	2.62	23	221	—
5.0 TO 5.2	5.1	3.35	2.79	23	226	—
5.2 TO 5.4	5.3	3.55	2.96	23	230	—
5.4 TO 5.6	5.5	3.75	3.12	23	235	—
5.6 TO 5.8	5.7	3.95	3.29	23	239	0.011
Greater than 5.8	6.0	4.25	3.54	23	245	0.011

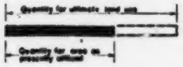


TOTAL – 122.2 – RAINFALL EVENTS PER YEAR

NOTES

1. OVERFLOW = 0.83 x RUNOFF
2. OVERFLOW IS IN INCHES FROM TRIBUTARY AREA.
3. VALUES DERIVED FROM ROSCOE STREET OVERFLOW ANALYSIS.
4. AMOUNTS OF 5-DAY BOD AND SETTLEABLE SOLIDS ARE FROM 19,000 ACRE AREA

LEGEND


Quantity for ultimate land use
Quantity for area as presently utilized

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

QUANTITIES OF POLLUTION TO UNDERGROUND RESERVOIR

HARZA ENGINEERING COMPANY DWG.NO. 387 P
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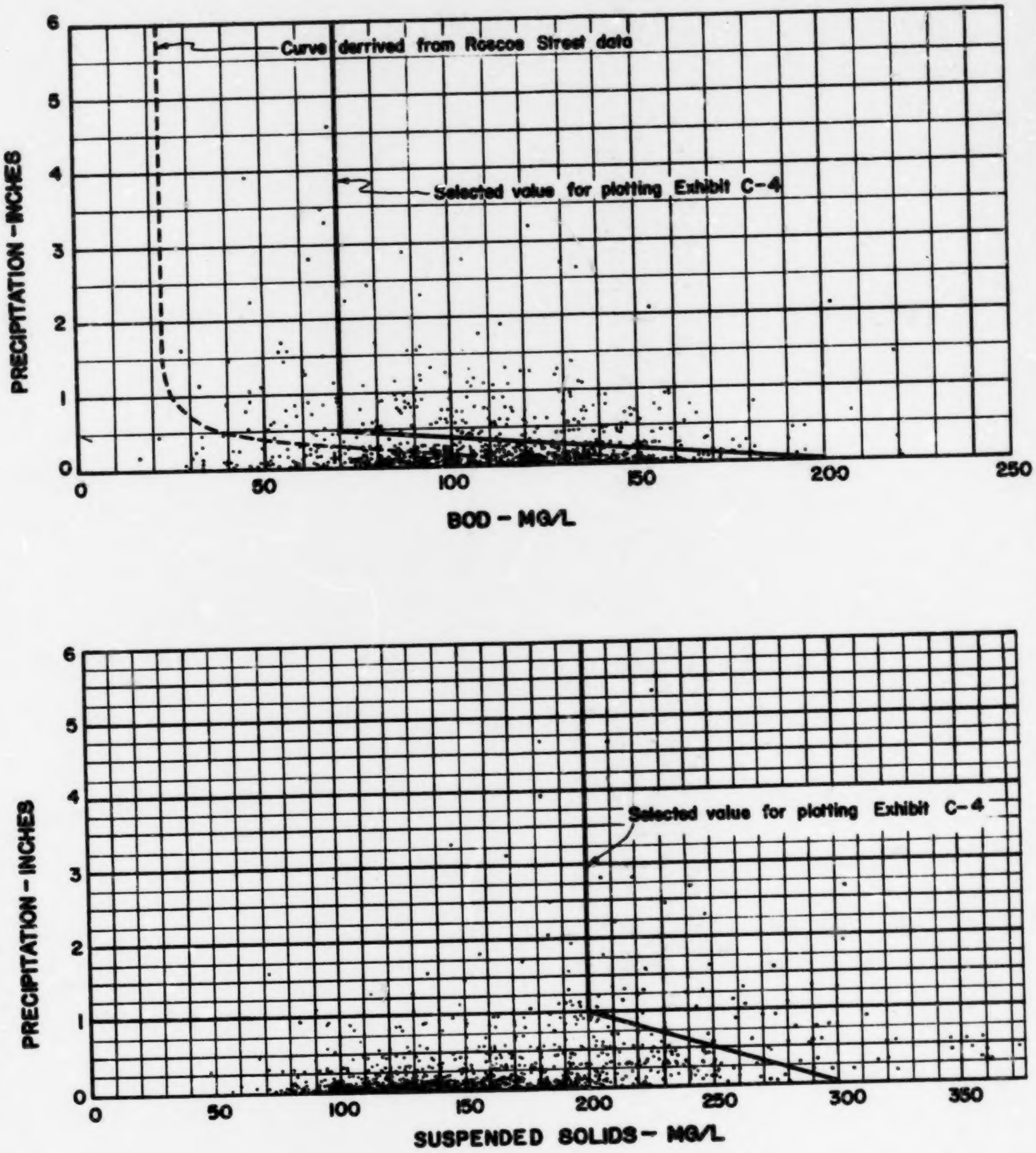
EXHIBIT C-3



NOTES:

1. *Calumet Sewage Treatment Works sewage characteristics 1954-1965 April thru November.*
2. *Values for precipitation less than 0.02" not plotted.*

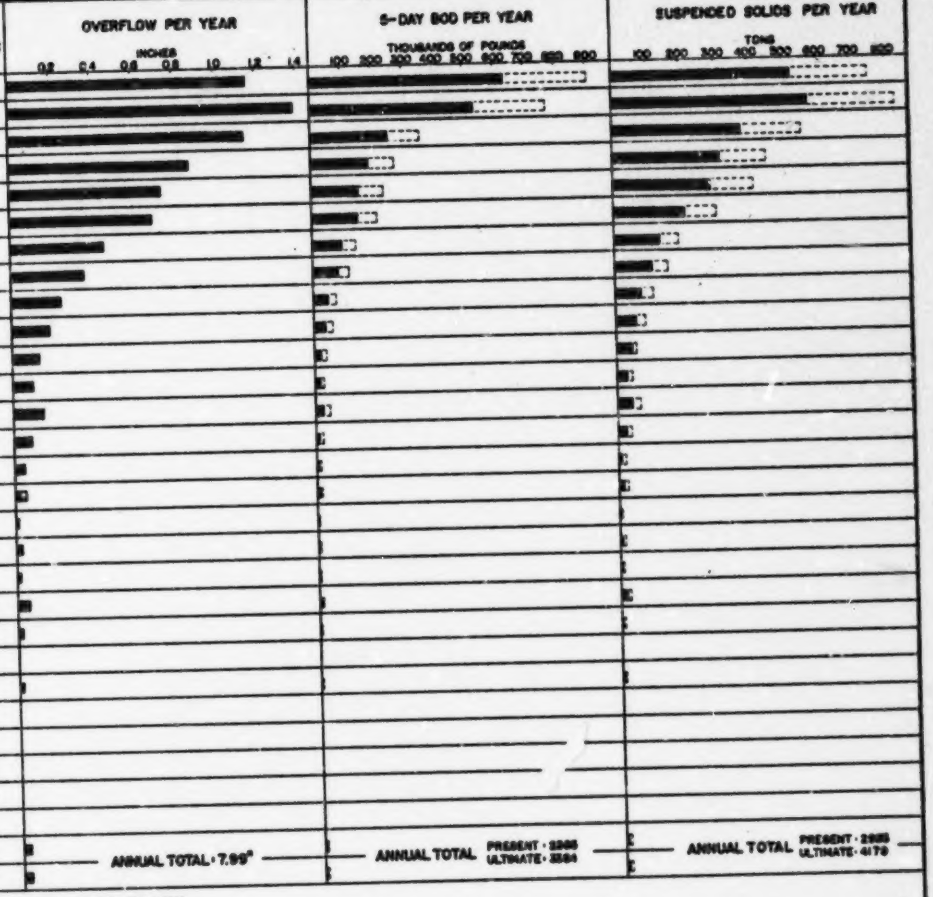
CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

QUALITY OF COMBINED SEWER OVERFLOWS

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT NO. 6524

EXHIBIT C-4

RAINFALL INTERVAL INCHES	AVERAGE RAINFALL INCHES	AVERAGE RUNOFF INCHES	AVERAGE OVERFLOW INCHES	AVERAGE 5-DAY BOD MG/L	AVERAGE SUSPENDED SOLIDS MG/L	NUMBER OF EVENTS
0.0 TO 0.2	0.1	0.03	0.06	175	290	78
0.2 TO 0.4	0.3	0.09	0.07	125	270	10.8
0.4 TO 0.6	0.5	0.15	0.12	70	250	9.6
0.6 TO 0.8	0.7	0.21	0.17	70	230	6.2
0.8 TO 1.0	0.9	0.28	0.23	70	210	3.2
1.0 TO 1.2	1.1	0.35	0.29	70	200	2.4
1.2 TO 1.4	1.3	0.42	0.34	70	200	1.3
1.4 TO 1.6	1.5	0.49	0.41	70	200	0.92
1.6 TO 1.8	1.7	0.56	0.47	70	200	0.53
1.8 TO 2.0	1.9	0.63	0.53	70	200	0.36
2.0 TO 2.2	2.1	0.70	0.58	70	200	0.22
2.2 TO 2.4	2.3	0.78	0.64	70	200	0.14
2.4 TO 2.6	2.5	0.87	0.72	70	200	0.21
2.6 TO 2.8	2.7	1.00	0.83	70	200	0.106
2.8 TO 3.0	2.9	1.18	0.98	70	200	0.053
3.0 TO 3.2	3.1	1.35	1.12	70	200	0.053
3.2 TO 3.4	3.3	1.55	1.29	70	200	0.011
3.4 TO 3.6	3.5	1.75	1.46	70	200	0.021
3.6 TO 3.8	3.7	1.95	1.63	70	200	0.011
3.8 TO 4.0	3.9	2.15	1.79	70	200	0.032
4.0 TO 4.2	4.1	2.35	1.96	70	200	0.011
4.2 TO 4.4	4.3	2.55	2.13	70	200	—
4.4 TO 4.6	4.5	2.75	2.29	70	200	0.011
4.6 TO 4.8	4.7	2.95	2.46	70	200	—
4.8 TO 5.0	4.9	3.15	2.62	70	200	—
5.0 TO 5.2	5.1	3.35	2.79	70	200	—
5.2 TO 5.4	5.3	3.55	2.96	70	200	—
5.4 TO 5.6	5.5	3.75	3.12	70	200	—
5.6 TO 5.8	5.7	3.95	3.29	70	200	0.011
Greater than 5.8	6.0	4.25	3.54	70	200	0.011

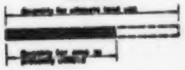


TOTAL – 122.2 – RAINFALL EVENTS PER YEAR

NOTES

1. OVERFLOW = 0.83 x RUNOFF
2. OVERFLOW IS IN INCHES FROM TRIBUTARY AREA.
3. VALUES DERIVED FROM CALUMET SEWAGE TREATMENT WORKS INFLUENT ANALYSIS
4. AMOUNTS OF 5-DAY BOD AND SUSPENDED SOLIDS ARE FROM 13,000-ACRE AREA.
5. MULTIPLY SUSPENDED SOLIDS BY 0.5 TO ESTIMATE SETTLEABLE SOLIDS.

LEGEND



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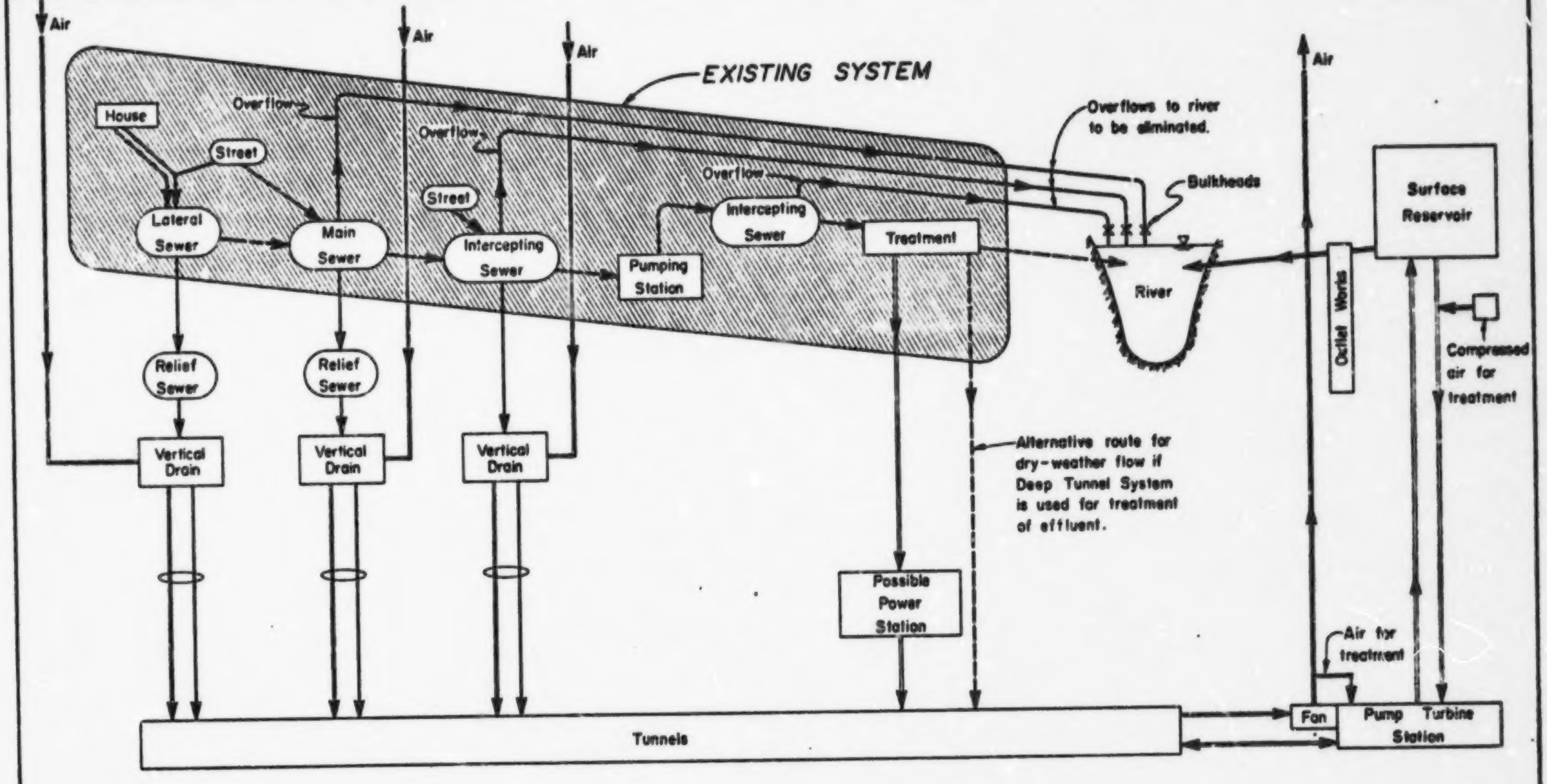
CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

QUANTITIES OF POLLUTION TO UNDERGROUND RESERVOIR

EXHIBIT C-5


Air
Air
Air
EXISTING SYSTEM
House
Street
Overflow
Overflow
Street
Overflow
Lateral Sewer
Main Sewer
Intercepting Sewer
Pumping Station
Intercepting Sewer
Treatment
Overflows to river to be eliminated.
Bulkheads
River
Air
Surface Reservoir
Outlet Works
Compressed air for treatment
Relief Sewer
Relief Sewer
Vertical Drain
Vertical Drain
Vertical Drain
Alternative route for dry-weather flow if Deep Tunnel System is used for treatment of effluent.
Possible Power Station
Air for treatment
Fan
Pump Turbine Station
Tunnels

LEGEND:
- Storm Flow
- Dry Weather Flow
- Power Generating Flow
- Air Flow

HARZA ENGINEERING COMPANY DWG. NO. 387P
BAUER ENGINEERING, INC. PROJECT NO. 6834

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

DEEP TUNNEL PLAN AS A TREATMENT PROCESS

Appendix D

AQUIFER PROTECTION

Synopsis

Protective measures must be taken to assure that pollution from the proposed underground reservoir will not contaminate the extensively used Cambrian-Ordovician Aquifer. The excavation is to be located in the Galena-Platteville Dolomite member of this aquifer system.

Results are presented herein of studies which were made to determine the effects of the underground reservoir upon the aquifer and to define the measures needed to assure that the aquifer could be protected from pollution resulting from the underground reservoir operations. The complex aquifer setting and characteristics were analysed by an electric analog computer which showed that ground water inflow during construction should not exceed about 4.5 million gallons per day (mgd), and would decline to about 2.0 mgd as water levels are lowered by regional pumpage and tunnel seepage. The studies found that aquifer protection could be assured by the installation of recharge wells to maintain aquifer water levels above the tunnel openings. Analog computer analysis showed that this protection would require recharge at rates gradually increasing from 1.0 mgd in the year 1980 to 8.5 mgd in the year 2010. Recharge facilities would utilize drinking quality injection water and would cost an estimated $2,600,000 for construction.

Aquifer Setting

Geology

The relationship of ground water to geology in Northeastern Illinois has been the subject of several studies. The most recent is that of

Suter[1] which has been summarized below. Geologic nomenclature and characteristics are discussed in Appendix B.

Regional ground water development is governed by geologic conditions, which, in general, have resulted in four essentially independent aquifer systems. These aquifer systems are, in upward sequence, the Mt. Simon, Cambrian-Ordovician, Silurian, and Glacial Drift. The lithologic and water-yielding properties of these systems are shown by Exhibit B-3.

The Mt. Simon Aquifer occurs immediately above the basement rock and consists of the Mt. Simon Sandstone and lower Eau Claire Formation. Impermeable zones of the middle and upper Eau Claire separate the Mt. Simon and Cambrian-Ordovician Aquifer. The Cambrian-Ordovician Aquifer overlies the upper Eau Claire Formation and consists of the Ironton-Galesville Sandstone, Franconia Formation, Trempeleau Dolomite, Prairie du Chien Group, Glenwood-St. Peter Sandstone and Galena-Platteville Dolomite. The consistently uniform grain size and poor cementation of the Ironton-Galesville Sandstone help to make this the most important regional aquifer. The relatively impermeable Maquoketa shale overlies and partially separates the Cambrian-Ordovician Aquifer from the Siluvian and Glacial Drift aquifer systems. Hydraulic interconnection of units of the Cambrian-Ordovician Aquifer is restricted by shaley and dolomitic intervening beds. The Silurian or Shallow Dolomite Aquifer overlies the Maquoketa shale and consists of the Alexandrian and Niagaran dolomites. The shaley zone at the base of the Niagaran locally restricts water movement between the Niagaran and Alexandrian dolomites.

The three bedrock aquifers are continuous over most of the region.

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[1] "Preliminary Report on Ground-Water Resources of the Chicago region, Illinois," by Max Suter et al. Illinois State Water Survey. Cooperative Ground-Water Report 1. 1959.

Productive sand and gravel deposits of the Glacial Drift Aquifer are not continuous throughout the region, but occur at different levels in the drift over the Shallow Dolomite Aquifer and sometimes are hydraulically interconnected with the Shallow Dolomite Aquifer.

Ground water occurrence and quality is governed by the characteristics and distribution of the geologic formations forming the ground water reservoir. The units of concern, in the immediate study vicinity, are those comprising the Cambrian-Ordovician Aquifer and overlying formations. Deeper units are effectively separated from overlying aquifers by the relatively impermeable Eau Claire Shales. All of the bedrock units tend to become less permeable toward the south and east.

Hydrology

Historically, ground water in bedrock aquifers is reported (Suter, et al[1/]) to have been recharged principally by precipitation to the drift in areas 50 miles west of Chicago, and then by leakage to successively deeper zones. Before man's development, movement was eastward and southward to areas of discharge by seepage and upward leakage through the Maquoketa Shale to rivers and lakes in areas south and east of Chicago. Pumpage, initiated about 100 years ago, has altered this condition so that movement is now largely toward six major centers of pumpage, and recharge of bedrock aquifers by downward leakage through the Maquoketa Shale has been greatly increased.

This movement is governed by the amounts and location of pumpage and the hydraulic characteristics of the Cambrian-Ordovician Aquifer and the Maqueketa Shale. Records and special studies by the Illinois

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[1/] Ibid.

Water Survey have provided an unusual body of data and information about both topics.

Total regional ground water pumpage from all aquifer systems is reported to have increased from 0.2 million gallons per day in 1864 to over 200 mgd in 1962 (Sasman, [2]). In 1962 over 50 mgd was withdrawn from the Cambrian-Ordovician Aquifer (Exhibit D-1) and 74 mgd was pumped from the Shallow Dolomite Aquifer. The remaining pumpage was from the Mt. Simon and Glacial Drift Aquifers. In 1966 pumpage is estimated to be about 10 mgd from the Cambrian-Ordovician Aquifer and 3.5 mgd from the Silurian Dolomite Aquifer in the study area and adjoining townships (excluding Indiana). There is no known significant pumpage from the Mt. Simon or Glacial Drift Aquifers in this area. The Silurian Dolomite Aquifer does discharge some water to deeper zones through wells uncased in both the Silurian Dolomite and Cambrian-Ordovician Aquifers at several locations in southeastern Chicago and northwestern Indiana.

Cambrian-Ordovician Aquifer hydraulic characteristics have been reported by Suter, et al[1], and Walton and Csallany[3]. Study of available data indicates that the following coefficients would apply in the project area.

	HYDRAULIC CHARACTERISTICS		
Aquifer	Transmissibility	Vertical Permeability	Storage
Galena-Platteville	300 gpd/ft	----------	0.01
Glenwood-St. Peter	2,500 gpd/ft	12 gpd/sq.ft	.0016
Prairie du Chien, etc.	----------	0.01 gpd/sq.ft	------
Ironton-Galesville	9,000 gpd/ft	50 gpd/sq.ft	.0016

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[2] "Ground-Water Pumpage in Northeastern Illinois through 1962," by Robert T. Sasman. Illinois Water Survey. Report of Investigation No. 50. 1965.

[1] Ibid.

[3] "Yields of Deep Sandstone Wells in Northern Illinois," by W. C. Walton and Sandor Csallany. Illinois State Water Survey. Report of Investigation 43. 1962.

Fewer wells have been drilled in the study area than in most other areas in the Chicago region, and no great accuracy is inferred for the coefficients. Formation transmissibility decreases rapidly toward the east in the vicinity of the study area, and additional field testing is necessary to refine the estimates of hydraulic characteristics.

There is very little data available to determine the hydraulic characteristics of the Silurian Dolomite, Glacial Drift, and Mt. Simon Aquifers in the project area. Available information (Csallany and Walton[4/]) indicates that the Silurian Dolomite Aquifer may have an average transmissibility of the order of 5,000 to 10,000 gpd/ft. in the study area. Locally, the dolomite may vary from moderately productive to virtually impermeable, because the flow of water occurs mainly through joints and crevices which are irregularly distributed and interconnected both vertically and horizontally. These openings are not always encountered by well drilling efforts. They can also afford channels for extensive and unpredictable movement of contaminants through the aquifer.

There is little data available concerning ground water quality in the study area. The deep sandstone aquifers all become progressively more highly mineralized toward the east, and water in the Mt. Simon Aquifer has a high chloride concentration in the study area. Analyses of water from deep sandstone wells (Wells No. 1, 2, and 4, Table I) indicate that these waters may be moderately mineralized, high in dissolved iron, hard and moderately high in chlorides. Little data is available for Silurian wells, but, locally, these waters may have a hardness and dissolved solid content, less than deep sandstone waters (Well No. 3, Table I).

[4/] "Yields of Shallow Dolomite Wells in Northern Illinois," by Sandor Csallany and W. C. Walton. Illinois State Water Survey. Report of Investigation 46. 1962.

The ground water occurs in a biologically pure state in aquifers, although instances of local pollution can occur where unclean materials or waste is allowed to enter the wells.

Water levels in the Cambrian-Ordovician Aquifer reflect a decline from positions which caused artesian flow in 1864 to depths of 600 feet below surface in 1964 (Exhibit D-2). The decline has been caused by increased discharge from the aquifer and the flow pattern resulting from the prevalent hydraulic characteristics of the formations and recharge conditions. Cambrian-Ordovician water levels are currently at an average elevation of about 140 feet msl., in the project area. They are declining at an average rate of from 10 to 20 feet per year, and would be, on the average, expected to fall below the top of the Galena-Platteville Dolomite in many parts of the Chicago Region by about 1980 (Walton[2]). Water levels in the Mt. Simon Aquifer are not know locally.

Silurian Dolomite and Glacial Drift water levels vary widely, but these levels average about 10 feet below surface in most of the project area.

Effects of Underground Reservoir

Seepage Inflow

The Reservoir location is proposed for the Galena-Platteville Dolomite in an area where ground water pressure levels could be over 500 feet above the bottom of the excavation. Initial seepage inflow is potentially large, and localized highly permeable zones of joints and crevices may be encountered. However, the average transmissibility is not high (Walton and Csallany[3/]) and construction treatment is expected to substantially reduce inflow from local joints or permeable areas.

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[2] Ibid.

The complex nature of ground water conditions precludes a mathematical solution of the amount of seepage inflow. Seepage has been estimated by means of an electric analog computer as discussed below.

Previous tunneling may also indicate the extent of seepage although tunnels have not been constructed in the Galena-Platteville Dolomite. Tunnels recently have been completed in the Silurian Dolomite along 79th Street. Leakage is reported to have been 2.3 cfs for the 4.6 mile long tunnel, just prior to completion. Ground water is believed to have been under about 180 feet of head. Dewatering pumpage is probably about 150,000 gallons per day for Stearns Quarry which is an open excavation about 1/4 mile square in the Silurian Dolomite and extends nearly 350 feet below the surrounding water-level surface. The average permeability of the Silurian Dolomite at these locations is probably about 10 times that of the Galena-Platteville Dolomite at the project site.

Pollution Hazard

After about 1980, water levels will begin to fall below the top of the Galena-Platteville Dolomite because of seepage into the underground reservoir and regional water-level declines (Exhibit D-3) and, presumably, below the top of the upper reaches of the underground reservoir. Thereafter, polluted water could enter the Cambrian-Ordovilian Aquifer from the underground reservoir unless some protection is provided.

Pollution must be excluded from this important source of uncontaminated water. Several alternative methods have been considered for

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[3/] Ibid.

this function, including lining of the underground reservoir and injecting uncontaminated water through recharge wells into adjacent and underlying zones. Lining costs may be prohibitive, and known forms of lining seem unlikely to be reliable in the dolomite aquifer. Bedding characteristics of the rocks and probable sealing problems with the injection piping would restrict effectiveness of an injection system located in the underground reservoir. However, if treated effluent is used for recharge, it may be most economical to place the distribution pipes in the reservoir.

The recommended means of aquifer protection is to use recharge wells to inject water at locations in the study area, with input rates sufficient to maintain water levels above the top of the Galena-Platteville Dolomite throughout the underground reservoir area. Thus, a pressure gradient would be established, maintaining flow always into the reservoir to prevent any polluted water from entering the aquifer. Monitoring wells are planned to verify the effectiveness of the proposed recharge well system.

The electric analog computer was used to determine the required rates of recharge, recognizing that injection rates would have to maintain the desired pressure levels and compensate for losses of injected water to adjacent and underlying formations and to the underground reservoir.

Electric Analog Computer

Introduction

The ground water setting is very complex, requiring construction of an electric analog computer to assess the effects of underground reservoir seepage and recharge wells. Analog computers permit rapid analysis of various complex ground water problems by means of the analogy that exists between electrical and aquifer systems. Such an analog computer consists of a network of electrical capacitors and resistors (analog model), a power supply, a waveform generator, a pulse generator, and an oscilloscope (excitation-response apparatus) (Karplus[5], Walton and Prickett[6]).

The resistors simulate aquifer transmissibility and aquitard permeability, and the capacitors store electro-static energy in a matter analogous to the storage of water in aquifers. The pulse and waveform generators energize the resistor-capacitor network, and the oscilloscope measures resulting energy levels throughout the network. Output oscilloscope traces (time-voltage graphs) are cataloged to provide data for the construction of water-level change maps and to determine the effects of discharge or recharge.

Analog Model

Aquifer characteristics and geometry have been simplified and modified to facilitate model construction, and pumpage has been assumed to be concentrated at the six major centers shown. The analog model for simulating the Cambrian-Ordovician Aquifer in northeastern Illinois consists of three horizontal, regular arrays of resistors and capacitors interconnected by two vertical, regular arrays of resistors. The model contains approximately 9,500 resistors and 4,000 capacitors. The bottom horizontal array of resistors and capacitors simulates the Ironton-Galesville Sandstone. Resistors have been connected vertically to terminals of the bottom horizontal array to simulate the aquitard units consisting of the Prairie-du-Chien Group, Trempeleau Dolomite, and Franconia Formation. The second horizontal array of resistors and capacitors simulates the

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[5] Analog Simulation, by W. J. Karplus. McGraw-Hill Book Company. 1958.

[6] "Hydrogeologic Electric Analog Computers," by W. C. Walton and T. A. Prickett. Journal Hyd. Div. ASCE. 1963.

Glenwood-St. Peter Sandstone. Resistors have been connected vertically to terminals of the second horizontal array to simulate the semi-permeable (aquitard) portions of the Galena-Platteville Dolomite. The third horizontal array of resistors and capacitors simulates the permeable aquifer portions of the Galena-Platteville Dolomite. Appropriate nodes of the third horizontal array have been connected to a wire to simulate the underground reservoir in the Lake Calumet area. The Maquoketa Shale and shallower deposits overlying the Cambrian-Ordovician Aquifer have not been included in the model because water levels by 1970 will be below these rocks in most areas of northeastern Illinois (Walton[7]), and the underground reservoir seepage and the recharge scheme will not materially change the movement of water in these rocks.

The model has been assembled at three horizontal scales. Terminals are located on a one-inch square pattern, with a scale of 1 in. = 2,000 ft in the study area, 1 in. = 20,000 ft. in the intermediate zone around the study area, and 1 in. = 40,000 ft. in most of the Chicago area. Greater detail was desired in the study area and progressively less detail was necessary at greater distances.

The model was operated on the premise that ground water flow in the aquifer units is essentially two dimensional and obeys the finite-difference form of the partial differential equation governing the non-steady state two-dimensional flow of ground water in an infinite, isotropic, and homogeneous aquifer (Walton[6]). This requirement was met in the model by connecting four resistors and a capacitor to each interior terminal of the horizontal arrays with the capacitors secured to a ground wire. Barrier boundaries were simulated by open circuits and the recharge boundary by grounding appropriate nodes of the model representing the Galena-Platteville Dolomite.

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[7] "Future Water-Level Declines in Deep Sandstone Wells in Chicago Region," by W. C. Walton. Ground-Water, Volume 2, No. 1. 1964.

[6] Ibid.

Excitation Response Apparatus

The excitation-response apparatus consists ot Tektronix Type 160A Power Supply, Type 162 Wave Form Generator, Type 161 Pulse Generator, Type RM 561A Oscilloscope with a Type 3A1 Dual Trace amplifier and a Type 2B67 Time Base.

The reservoir seepage rate or well recharge rate was computed by incorporating a small resistor, R_1. in series in the circuit between the pulse generator and the analog model. The voltage drop across the calibrated resistor was measured with the oscilloscope to permit computation of the flow rates.

Analog Computer Results

The analog model was coupled to the excitation-response apparatus to become the electric analog computer (Exhibit D-4). The pulse generator was connected to the wires simulating the underground reservoir, with the output adjusted in accordance with water-level data (Walton[7/]), shown in Exhibit D-5. Seepage into the reservoir was estimated for the varying artesial pressure conditions in the Cambrian-Ordovician Aquifer. It was assumed that the sloping underground reservoir would be at an average depth of 150 feet below the top of the Galena-Platteville Dolomite.

As shown in Exhibit D-5, the artesian pressure is expected to decline from an average position of about 200 feet above the top of the Galena-Platteville Dolomite in 1966 to the top of the Group in 1980 as the result of

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[7/] *Ibid.*

the estimated regional water-level declines and withdrawals from the underground reservoir. Artesian pressure declines, in the vicinity, resulting from withdrawals from the underground reservoir, were estimated by connecting the oscilloscope to terminals representing observation points.

The analog computer results show that average seepage into the underground reservoir would probably decline from about 4.5 mgd for 1966 piezometric levels to 2.0 mgd in 1980 (Exhibit D-6) when water levels are expected to reach the top of the Galena-Platteville Dolomite. Analysis was based on the assumptions that: (1) the Galena-Platteville Dolomite has uniform permeability, and (2) the inflow is uniformly distributed along the underground reservoir length. Lower sections of the underground reservoir and those sections intersecting more permeable areas are expected to have higher than average inflow rates. Higher sections of the underground reservoir and those intersecting dense-impermeable rock areas will have correspondingly lower inflow rates. Inflow could also be reduced by grouting, of locally permeable areas, as a construction expedient.

The computer results show that recharge requirements will gradually increase from 1 mgd in 1980 to 8.5 mgd in 2010, to maintain water levels above the top of the Galena-Platteville Dolomite. Based on successive trials using the computer, the number of required recharge wells terminating in the Glenwood-St. Peter Sandstone will increase from one in 1980 to 17 in 2010. The recharge wells would inject at average rates of 0.5 mgd each. There would be four injection wells spread 2000 feet apart at each of the four corners of the underground reservoir area and one injection well near the center of the area. Higher injection rates could be obtained by wells terminating in the

Ironton-Galesville Sandstone, but the high permeability of this unit would increase substantially the total injection requirement. Termination of the recharge wells in the Galena-Platteville Dolomite would restrict injection to very low rates and greatly increase the required number of recharge wells.

The analog computer also was used to determine the effects of recharging upon the water levels at the principal center of pumpage from deep sandstone wells. Water levels are expected to decline less in deep sandstone wells in northeastern Illinois as the result of the artificial recharge scheme. For example, by 2010 water supply wells at Elmhurst would be about eight feet higher than they would have been without artificial recharge. Estimated effects by 2010 at major pumping centers in northeastern Illinois are given below:

RECHARGE EFFECTS AT PUMPING CENTERS

Pumping Center	Water Level Effect*
Chicago (Summit)	18 feet
Elmhurst	8 feet
Des Plaines	6 feet
Joliet	5 feet
Elgin	2 feet
Aurora	2 feet

These estimates of water-level effects do not have a high degree of accuracy, but they are shown to indicate probable order of magnitude.

Based on the analog computer studies, it was concluded that the pollution hazard can be eliminated by artificial recharge of the Cambrian-

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* Reduction of water level decline due to proposed underground reservoir recharge system.

Ordovician Aquifer in the vicinity of the underground reservoir system. Recharge rates would not be physically excessive. The average seepage rate into the underground reservoir also would not be excessive. More extensive investigation will be made to determine the actual formation coefficients at the project site during feasibility studies for the underground reservoir system.

Aquifer Recharge

Recharge Operation

Recharge wells are proposed to maintain water levels in the Cambrian-Ordovician Aquifer at the top of the Galena-Platteville Dolomite to prevent pollution by the underground reservoir.

The underground reservoir is proposed for locations in the Galena-Platteville Dolomite. Ground water is stored and moves through secondary openings (crevices, joints, etc.) in such rock, and the nature of these openings is such that pollution can travel, unaltered, rather great distances. Thus, recharge has been proposed to maintain pressures throughout the area at levels above the openings, by distribution through both the Glenwood-St. Peter Sandstone and Galena-Platteville Dolomite. Monitor wells will be installed to verify the efficiency of this pressure distribution and to assure that pollution does not enter the aquifer.

Recharge will be carried out by injecting "drinking quality" water into the Glenwood-St. Peter Sandstone and the Galena-Platteville Dolomite. The studies discussed above showed that by 2010 about 8.5 mgd would be required. The injected water would be maintained at bacteriological standards adequate for drinking water and would not exceed the mineral content of water found in these formations at present (Table D-1).

TABLE D-1

WATER SAMPLE ANALYSES

Number	1	2	3	4
Owner	Interlaken Steel Company	American Malting	Norris Grain Elev. Comp.	Carnegie-Illinois Steel Corporation
Location	Sec. 13, T 37N R 14E	27, 37N 14 E	6, 37N, 15E	32, 38N, 15E
Date	1942	1914	1914	Aug. 21, 1942
Lab. No.	93755	27782	28532	93750
Depth	1690 ft	1648 ft	346 ft	1660 ft.
Turbidity				10. ppm
Color				0.
Odor				0.
Iron (total)	1.1 ppm	0.0 ppm	0.2 ppm	0.9
Manganese				0.0
Calcium				198.8
Magnesium				74.3
Ammonium				0.5
Sodium				230.
Silica				11.0
Chloride	200.	200.	30.	207.
Nitrate				1.1
Sulfate				757.8
Alkalinity	232	206	199	222.
Hardness	709	658	29	802.
Total Dissolved Minerals	1581. ppm	1518 ppm	296 ppm	1626. ppm

Recharge well construction would assure protection from pollution by surface waters. Injection water turbidity would be maintained at levels found to be most economical by balancing costs of treatment to reduce turbidity against costs of well rehabilitation to remove accumulated solids.

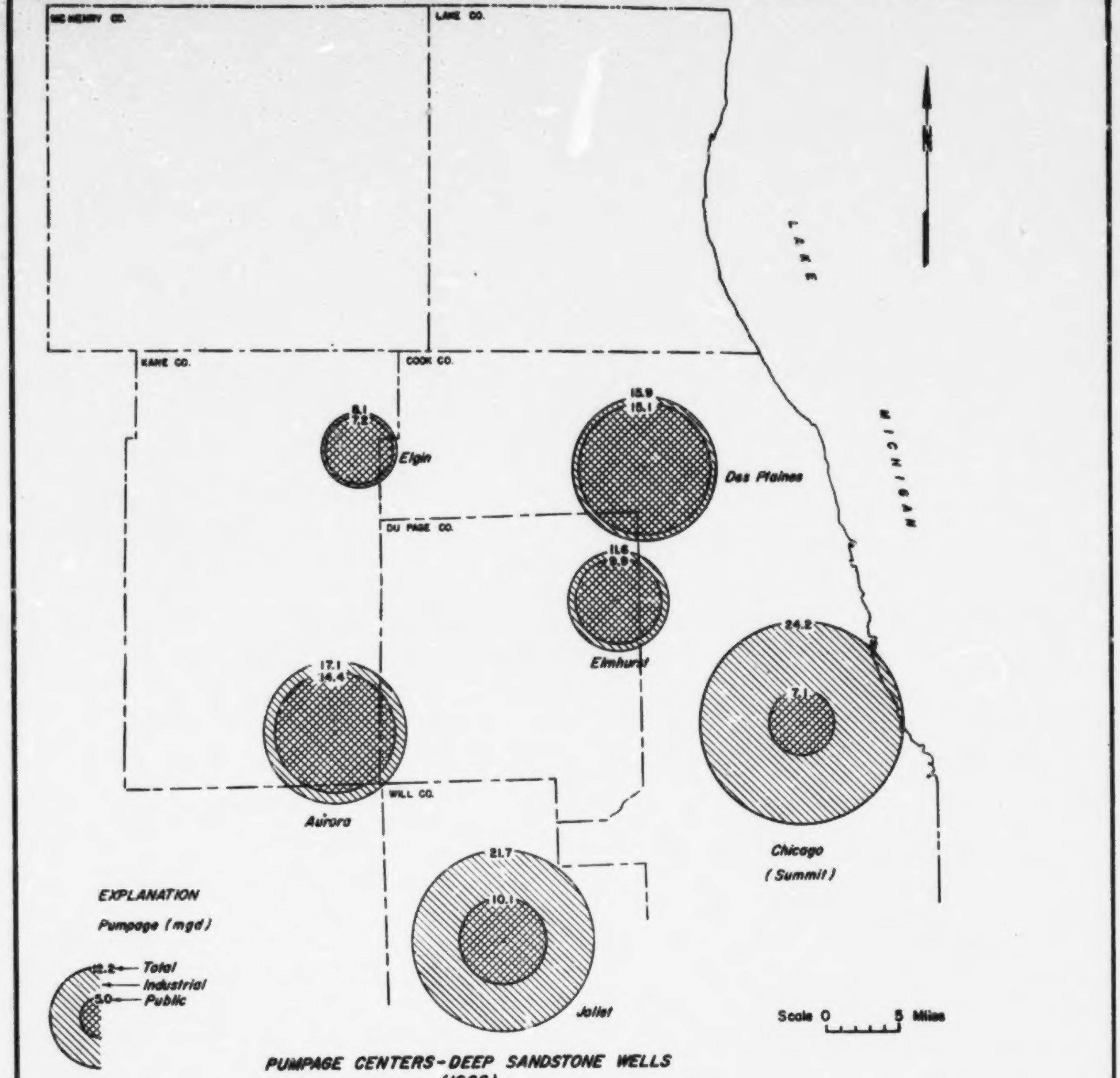
Recharge wells have been operating for several years, at various locations in California, injecting treated sewage to maintain aquifer water levels and recover fresh water. Cooling water which has been pumped from the Ironton-Galesville Sandstone is returned by a recharge well in the Glenwood-St. Peter Sandstone near Ottawa, Illinois. Other successful recharge operations could be cited; however, it still appears desirable to demonstrate the feasibility of recharge for conditions in the project area during the next phase of studies.

Recharge Cost

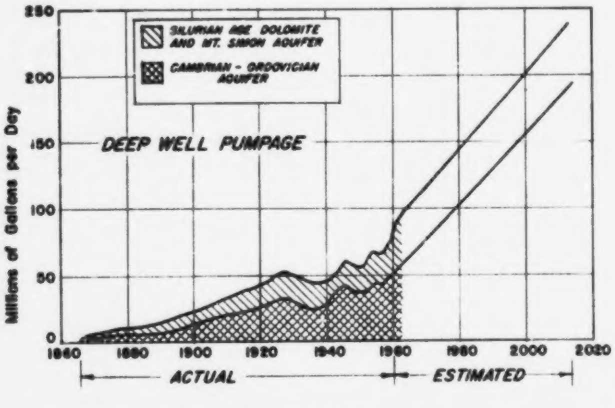
The level of this study and lack of detail about the source of injection water preclude detailed estimates of capital or annual costs. However, the following estimate is presented within those limitations, to establish the order of magnitude of this phase of the plan:

Injection Wells and Facilities	$ 581,000
Injection Water Distribution Mains	1,751,000
Miscellaneous Structures and Equipment	268,000
Total	$ 2,600,000

EXHIBIT D-1


MC HENRY CO.
LAKE CO.
KANE CO.
COOK CO.
DU PAGE CO.
WILL CO.
LAKE MICHIGAN
N
8.1
7.2
Elgin
15.9
15.1
Des Plaines
11.6
9.9
Elmhurst
17.1
14.4
Aurora
24.2
7.1
Chicago
(Summit)
21.7
10.1
Joliet
EXPLANATION
Pumpage (mgd)
12.2 Total
Industrial
5.0 Public
Scale 0 5 Miles

PUMPAGE CENTERS-DEEP SANDSTONE WELLS
(1962)



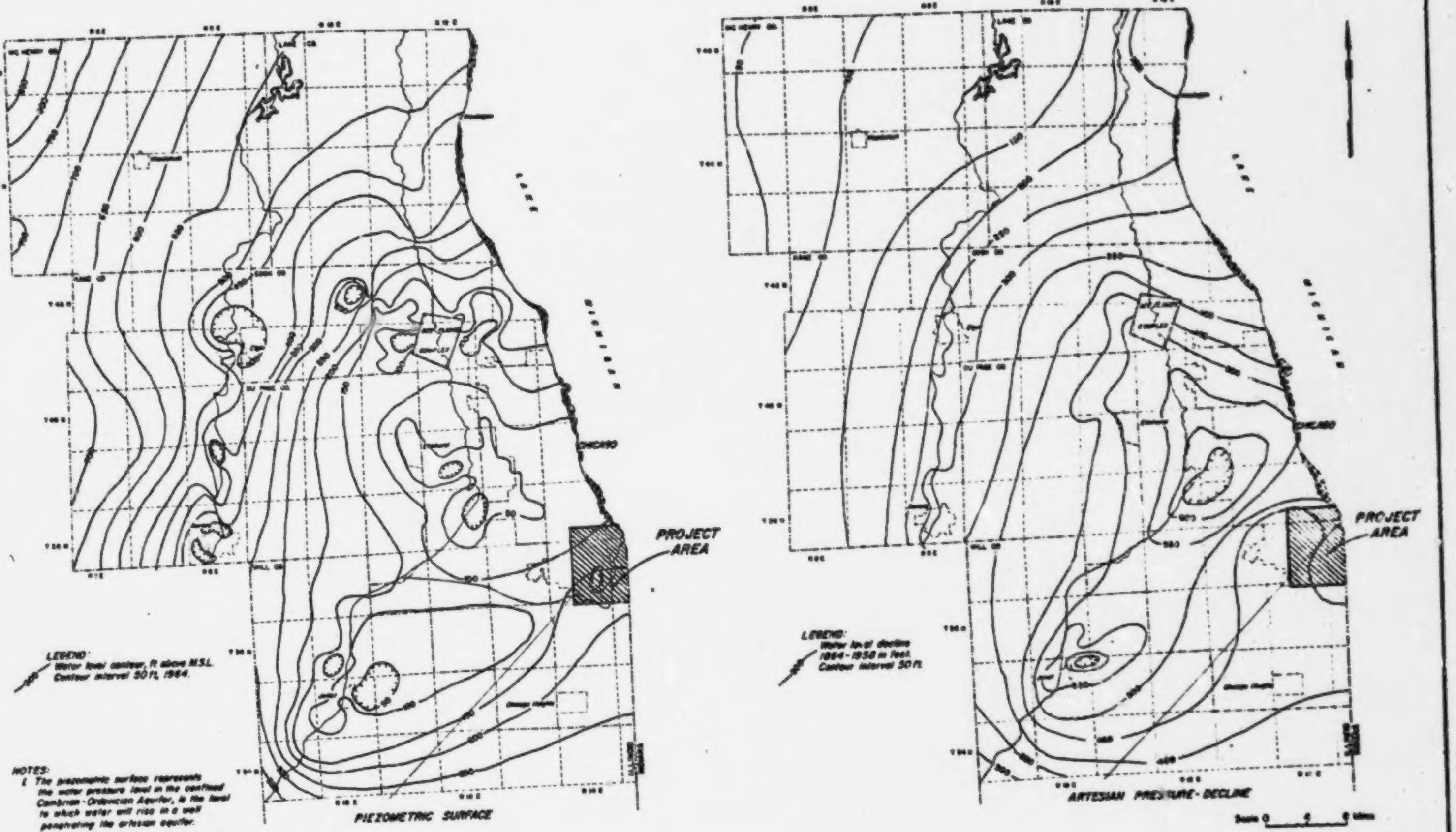
NOTE:
Drawings based on Illinois Water Survey; Walton 1964 and Sasman 1965.

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

PUMPAGE - DEEP AQUIFERS

HARZA ENGINEERING COMPANY DWG. NO. 387 P-41
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT D-2



PIEZOMETRIC SURFACE

ARTESIAN PRESSURE-DECLINE

LEGEND:
Water level contour, ft above M.S.L.
Contour interval 50 ft, 1964.

LEGEND:
Water level decline
1864-1958 in feet.
Contour interval 50 ft.

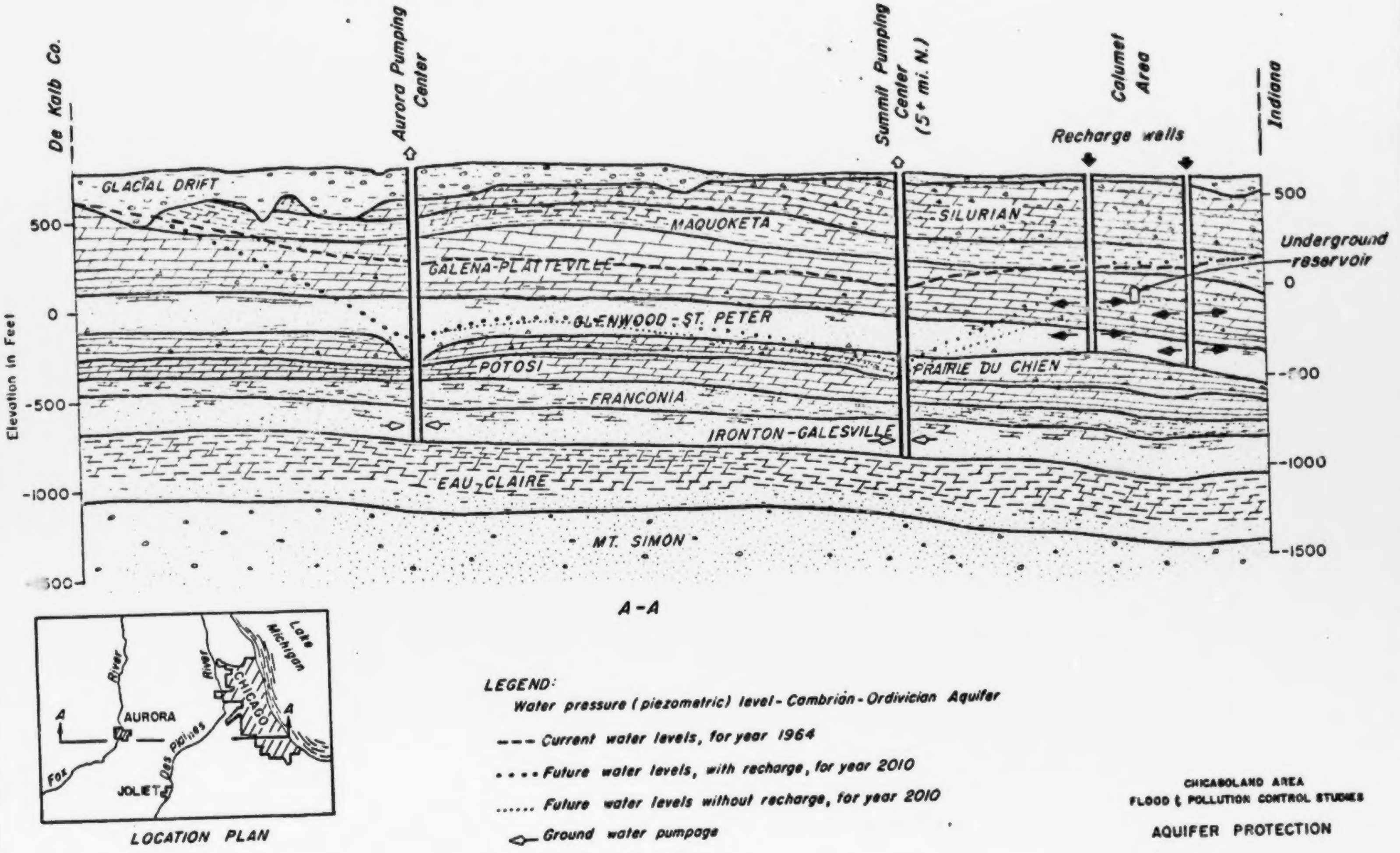
NOTES:
1. The piezometric surface represents the water pressure level in the confined Cambrian-Ordovician Aquifer, is the level to which water will rise in a well penetrating the artesian aquifer.
2. Drawings based on Illinois State Water Survey, Walton, 1964, and Sasman, 1965.

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

CAMBRIAN ORDOVICIAN AQUIFER WATER LEVEL MAPS

HARZA ENGINEERING COMPANY DWG. NO. 387 P-42
BAUER ENGINEERING, INC. PROJECT 6324

EXHIBIT D-3


De Kalb Co.
Aurora Pumping Center
Summit Pumping Center (5+ mi. N.)
Calumet Area
Recharge wells
Indiana
Underground reservoir
Elevation in Feet
500
0
-500
-1000
-1500
GLACIAL DRIFT
MAQUOKETA
SILURIAN
GALENA-PLATTEVILLE
GLENWOOD-ST. PETER
POTOSI
PRAIRIE DU CHIEN
FRANCONIA
IRONTON-GALESVILLE
EAU CLAIRE
MT. SIMON
Lake Michigan
River
AURORA
CHICAGO
Des Plaines
Fox
JOLIET
A

A-A

LOCATION PLAN

LEGEND:

Water pressure (piezometric) level - Cambrian-Ordivician Aquifer

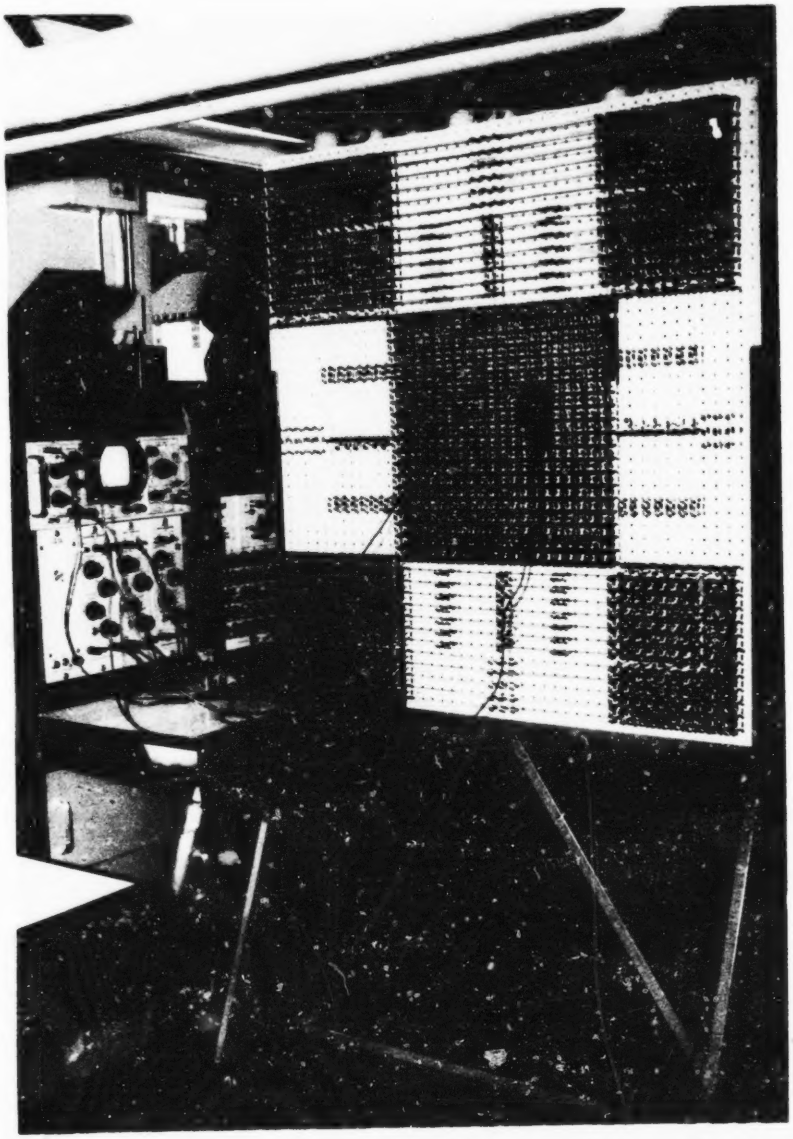
- - - - Current water levels, for year 1964
- • • • • Future water levels, with recharge, for year 2010
- ...... Future water levels without recharge, for year 2010
- ⇦ Ground water pumpage
- ⬅ Ground water recharge

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

AQUIFER PROTECTION

HARZA ENGINEERING COMPANY DWG. NO. 387P-4
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT- D-4

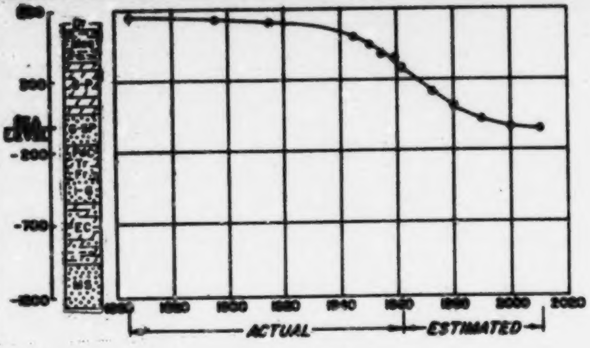


CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

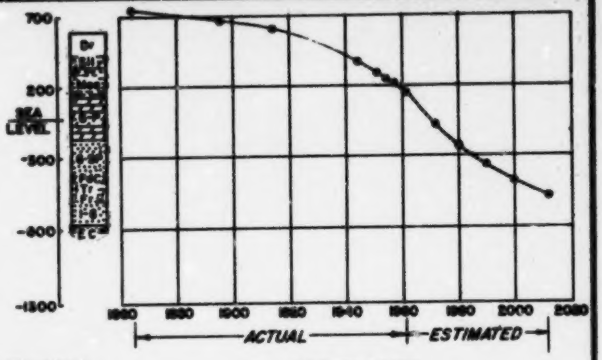
ELECTRIC ANALOG COMPUTER

HARZA ENGINEERING COMPANY DWG. NO. 387P-44
BAUER ENGINEERING, INC. PROJECT 6524

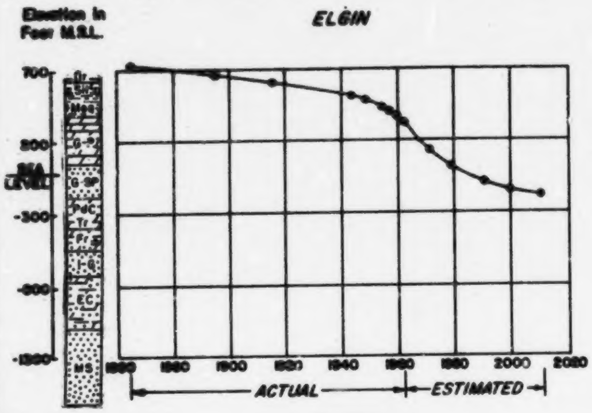
EXHIBIT D-5



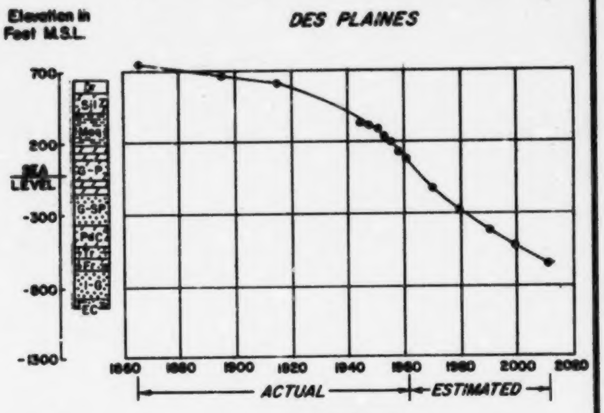
ELGIN



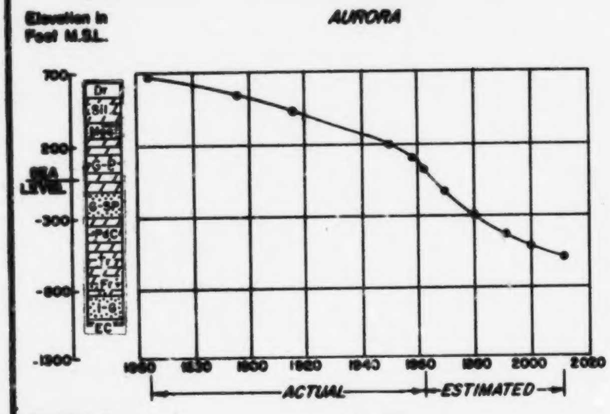
DES PLAINES



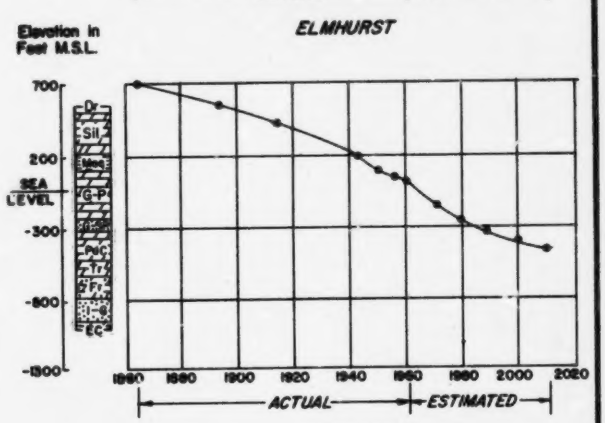
AURORA



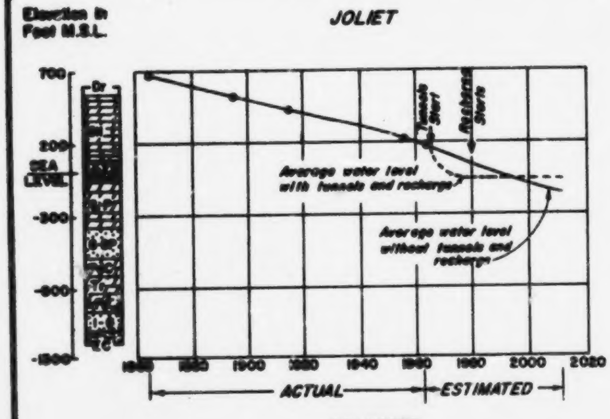
ELMHURST



JOLIET



CHICAGO
(Summit)



CALUMET

NOTES:

1. *Data after Illinois Water Survey, Walton, 1964.*
2. *Water levels at principal regional pumping centers, Cambrian - Ordovician Aquifer.*

CHICAGOLAND AREA

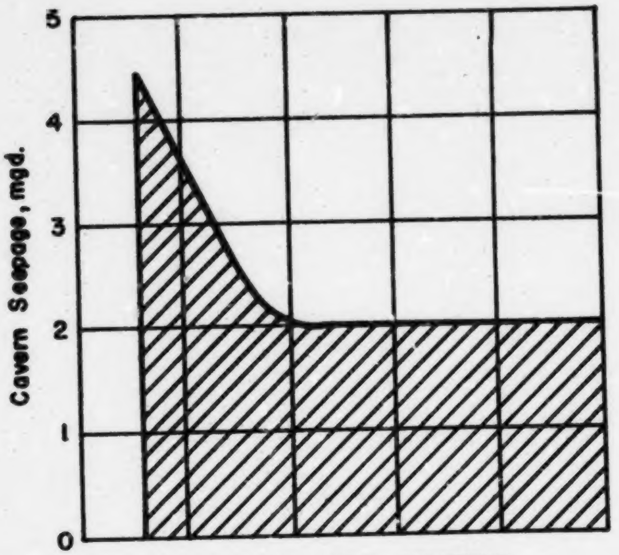
FLOOD & POLLUTION CONTROL STUDIES

CAMBRIAN ORDOVICIAN AQUIFER WATER LEVEL CHARTS

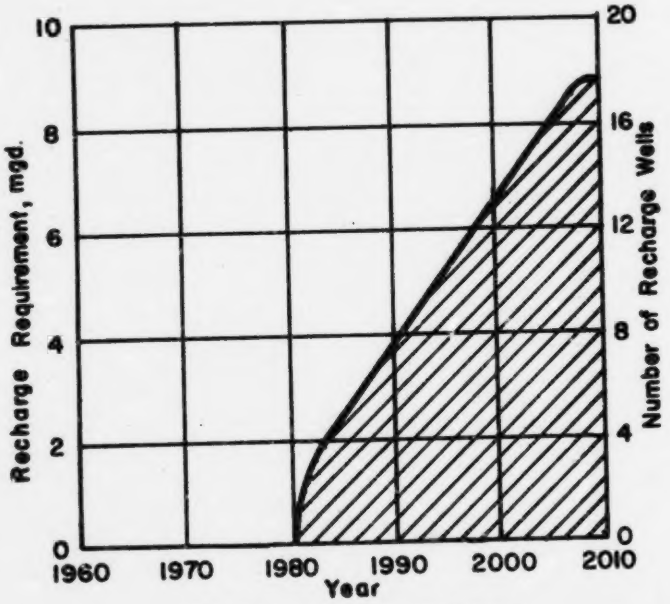
HARZA ENGINEERING COMPANY DWG. NO. 387P-45

BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT- D-6



SEEPAGE INTO CAVERNS



RECHARGE REQUIREMENT

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

TUNNEL SEEPAGE - RECHARGE

HARZA ENGINEERING COMPANY DWG. NO. 387P-46
BAUER ENGINEERING, INC. PROJECT 6524

Appendix E.

POWER

Role of Pumped-Storage

The Deep Tunnel Plan offers unique potential for the installation of a pumped-storage power station. The Chicago metropolitan area has a large electrical load which is now provided almost entirely by fossil fuel-fired and nuclear sources. Substantial advantages are available to such systems from hydroelectric installations, particularly when they can be located in such close proximity to the electrical load.

History of Pumped-Storage

Pumped-storage installations do not represent a new concept. The earliest pumped-storage plants were constructed about 1900. The first pumped-storage plant in this country was the Rocky River Plant near Milford, Connecticut, which was placed in operation in 1928. There were no further stations constructed in this country until the mid-1950's, although many plants were built throughout Europe, especially in Germany.

Since 1950, a number of pumped-storage projects have been built or have received active consideration. The concept of pumped-storage has become increasingly popular with the larger utilities of this country because of the advent of improved dual purpose pump-turbine units and the ever-increasing difference between peak and off-peak generation requirements.

The pumped-storage installations in the United States which are currently existing or under construction are shown in Table E-1.

TABLE E-1

PUMPED-STORAGE INSTALLATIONS IN THE UNITED STATES

Name	Location	Installed Capacity (Megawatts)	Year Completed or Scheduled
Constructed			
Buchanan	Texas	12.5	1950
Flat Iron	Colorado	8.5	1954
Hiwassee	Tennessee	59.5	1956
Lewiston	New York	240	1961
Taum Sauk	Missouri	375	1963
Smith Mountain	Virginia	120	1963
Yards Creek	New Jersey	338	1965
Under Construction			
Cabin Creek	Colorado	320	1967
Muddy Run	Pennsylvania	800	1968
San Luis	California	424	1968
Oroville	California	261	1969
Seneca	Pennsylvania	300	1969

Elements of a Pumped-Storage Installation

The principal elements of a pumped-storage installation include the following:

(a) An upper reservoir

(b) A lower reservoir

(c) A water conductor connecting the two reservoirs

(d) A dual purpose, reversible, pump-turbine unit, capable of pumping water or generating hydroelectric power

(e) Transmission lines connecting the installation to the power system.

The usual method of operation is to pump water from the lower reservoir to the upper reservoir during off-peak (generally night-time) hours, when surplus power generating capability exists in the power system. Water is then released from the upper reservoir to the lower reservoir during peak power load hours, generating hydroelectric power at a time when the remainder of the generating capability in the power system is normally taxed to near its capacity. Because of losses in efficiency, inherent in any pumping or generating equipment, about three kilowatt-hours of energy are used for every two kilowatt-hours produced in the pumping-generating cycle.

Factors Affecting Economics of Pumped-Storage

The difference in elevation between upper and lower reservoirs, or head, is a critical factor in the economics of pumped-storage installations. Except in unusual circumstances, pumped-storage plants will not be economical if the head is less than about 300 to 400 feet. The limiting head for which reversible pump-turbine units have been designed is about 1200 feet. For greater heads, separate pumping and generating units must be installed; therefore, pumped-storage installations with greater heads are usually not economically attractive. The range in head between 600 and 1000 feet usually combines the greatest economics and the most experience in design.

The length of water conductor connecting the two reservoirs is another important factor in pumped-storage economics. Potential developments which are otherwise attractive are frequently uneconomical because of the horizontal distance between upper and lower reservoirs, requiring long and costly water conductors.

The upper and lower reservoirs must be of sufficient capacity to accommodate all of the water released during the generating cycle. Depending on the operation of the remainder of the system, this cycle may be either daily or weekly. The cost of providing upper and lower reservoirs is also frequently critical to the economics of a pumped-storage facility.

The last element that is critical to economics is the length of transmission lines. Despite rapid improvement in method of transmission, there is a distinct advantage to a pumped-storage project located near the load center over one located at some distance. The cost of transmission lines and the attendant line losses can easily be determining factors.

Functions of a Pumped-Storage Installation

The most important functions in pumped-storage operations which have increased the usefulness of such installations are as follows:

(a) Power system economies introduced by generating high-value on-peak energy, as opposed to consumption of low-value off-peak energy during the pumping stage of operations.

(b) Economies realized by operation of thermal and nuclear generating stations at more uniform rates of production (higher load factor), thereby improving the efficiency of operation of such installations. The pump-storage facility contributes to this manner of operation in two ways: (1) by eliminating a portion of the peak load from the residual to be carried by thermal or nuclear sources and (2) by adding to the load during low-load hours (termed the "valley" of the load curves).

(c) Replacement of costly starting and stopping of steam turbines during periods of rapidly fluctuating power load. (Hydroelectric stations are capable of very rapid changes in rates of production.)

(d) Use as a ready reserve on the power system, to be started in emergency outages of other equipment. Pumped-storage installations can be brought "on the line" in about five minutes. The use of steam units for ready reserve is quite expensive, as they must be maintained in a non-productive, but "hot", condition, to be immediately available in emergencies.

(e) Hydroelectric generating units are known to be somewhat more reliable than thermal or nuclear units. As a general rule hydroelectric units are considered to have about 10 percent greater availability for generation than thermal units.

Chicagoland Utility Power Requirements

The Commonwealth Edison Company is the sole utility supplying electric power commercially in the Chicagoland area. Its present annual peak load is about 6,500,000 kilowatts, which is expected to increase to over 13,000,000 kilowatts by 1975. The present load is met almost entirely by thermal and nuclear sources. There is no potential for conventional hydroelectric power development within ready transmission distance of the Chicagoland area. Neither are there outstanding possibilities for the more ordinary type of pumped-storage installations near the area. Thus, the addition of a pumped-storage facility within the City of Chicago, as proposed in the Deep Tunnel Plan, has a natural potential.

Meetings were held with representatives of Commonwealth Edison Company to discuss their view of pumped-storage as related to their system. They feel that an installation of between 300 and 400 megawatts would be the minimum they could economically fit into their system operations. They expressed considerable interest in having a ready reserve in case of outage and in possible daily use throughout the year. between six and eight hours per day during the week days. The value of filling the "valleys" in the load curve by a pumping load is apparently gaining more favor as larger thermal and nuclear units are being constructed, since it allows the units to operate more efficiently at a higher load factor. Another possible use would be to pick up load until interchange power can be brought on line which requires about fifteen minutes.

All of the above purposes cannot be met concurrently. If the pumped-storage plant is used on a daily operational cycle,, it will not ba available as a ready reserve during the time it is generating. However, it can be used for that purpose during the pumping stage, as the pumping load can be dropped immediately, providing the equivalent of a ready reserve. Also, it can be used for daily operation during a portion of the year and as a stand-by for the remainder of the time.

Based on a 1975 peak load of 13,500 megawatts and the general situation in operation for utilities, at least 1350 megawatts could be in peaking capacity by pumped-storage facilities. There is unquestionably a market for pumped-storage power in such a system if the costs, initial and annual, are within the economic limits of alternative sources of power.

In the final analysis the degree of interest of the Commonwealth Edison Company will depend on the cost of peaking power from the pumped-storage installation. This view is a logical one and has been expressed by their officials. Allocations of costs of the various features of the Deep Tunnel Plan will not be made until the feasibility stage of investigations. Thus, there has been no opportunity, or need, of proceeding any further than determining the general interest of the utility in these prefeasibility studies.

The anticipated daily-load curve of the Commonwealth Edison Company for the peak day in 1975 is shown on Exhibit E-1. Also shown is a curve relating the kilowatt-hours of energy that must be generated during the peak load period of 1975 to match various amounts of peaking capacity.

Description of the Pumped-Storage Installation

General Arrangement

The pumped-storage installation proposed for the Deep Tunnel Plan includes utilization of about 800 feet of head between the upper and lower reservoirs. Reversible pump-turbines will serve the pumping and generating functions of the pumped-storage operation. In addition, the pumps will be utilized to evacuate storm water from the underground storage system.

If the project were designed for flood and pollution control only, a rather large amount of pumping capacity would be required to evacuate storm water. The addition of pumped-storage capacility can be at relatively low cost. The incremental cost of reversible pumping-generating machinery is only about 50 percent greater than that of pumping units, while all other costs, including powerhouse structures, miscellaneous mechanical and electrical equipment and water conductions, remain substantially the same.

Having introduced a pumped-storage installation into the scheme as a probable economical addition, it then becomes attractive economically to install as much pumped-storage capacity as can be utilized under the load curve of Commonwealth Edison, as limited by the storage available for supply of matching kilowatt-hours. This approach appears economical even though it calls for more matching pumping capacity than would be required for evacuation of storm water overflow. The installation of such surplus pumping capacity allows a reduction to be made in the amount of underground space to be provided for storage of storm water, thereby introducing overall economy to the Plan.

The proper balance between volume of underground storage, the allocation of storage to power purposes, pumping capacity, and hydroelectric generating capacity requires an involved economic study which could not be solved on an optimized basis for this report. It will undoubtedly require a systems analysis approach, making extensive use of digital computers. Such a study will be made in the feasibility investigations.

The selection of storage volumes for this report was based primarily on judgment, utilizing studies of possible conflicts between power and flood control, as described later. This, in turn, established the amount of kilowatt-hours which could be generated on peak and, from this, the justified installed generating capacity was determined.

The subsequent studies referred to indicated that about 40 percent of the lower reservoir volume could be utilized for normal operations for pumped-storage. Based on a total underground storage volume equivalent to a runoff of 2.5 inches of depth over the drainage area, a storage volume of about 1120 acre-feet would be usable for normal power operations. The same amount of usable storage would be provided in the upper reservoir. This would be sufficient to provide 750,000 kilowatt-hours of continuous generation (without any intervals of pumping water back). From the curve in the upper right of Exhibit E-1, it can be observed that this would support about 300,000 kilowatts of installed capacity.

Power Station Description

Station Layout. The power station layout is shown on Exhibit 9 following the Main Report. The main generator hall would have a total length of about 215 feet, a width of about 80 feet, and a maximum height of about 158 feet. Unit spacing would be about 50 feet. A concrete arch is planned for the roof. Access to the powerstation would be by means of two shafts, one for equipment and one for cables and personnel. The powerstation would have a service bridge crane of about 275-ton capacity for installing and servicing the units and valves.

In addition to the main units the powerstation would provide space for the transformers and draft tube gates, in chambers upstream and downstream of the powerstation, respectively. Additional space in the generator hall would provide for valves, switchgear, control equipment, ventilating equipment, water and drainage systems, personnel and visitors' facilities. In the erection and service areas, space would be provided for maintenance shops, storage batteries, air compressor and oil storage rooms, and for water and sewage equipment.

Equipment. The average project head, between the surfaces of the upper and lower reservoirs, is about 800 feet. The selected units would be rated at 100,000 kilowatts, of the vertical shaft, pump-turbine, motor-generator type. When pumping the units would operate in the opposite direction as that for generating. The selected units would be capable of meeting loads over a wide range.

Each of the three motor-generators would be connected to its individual three-phase, 125 mva transformer. The transformers would be housed in individual cubicles in the transformer gallery and would be provided with automatic fire protection facilities.

Water Conductors

Penstocks would connect the upper reservoir to the pump-turbine units. One penstock would service two units and would have a diameter of about 12 feet. The maximum velocity would be about 29 feet per second.

On the suction side of the pump-turbine the conduit would be approximately 11 feet in diameter and could be closed by means of a slide gate capable of taking full head hydrostatic loading. The gate would have a bonnet to protect the gallery and powerstation from flooding.

For a distance of approximately two hundred feet upstream and downstream of the powerstation, the conduits would be steel lined to protect against leakage due to internal water pressure. The conduits would be concrete backfilled where steel lined and concrete lined where not steel lined.

Upper Reservoir

The upper reservoir would be formed by a dike enclosing an area of about 135 acres. The dike would be about forty feet high, with side slopes of 2.5:1 on the inside and outside. The dike would be constructed of rockfill excavated from the underground reservoir and would have a core of local clay. Filters on either side of the core would be made from either excavated crushed rock or from local quarry stone.

The reservoir would contain approximately 2360 acre-feet of storage, of which 1120 acre-feet would be allocated to power. A spillway would be constructed in the south-east corner of the reservoir to discharge south of O'Brien Locks to avoid contamination of Lake Michigan.

Transmission

From the underground transformers, oil-filled cable will lead to the switchyard via the access shaft to breakers, disconnect switches, and reversing switches and the bus, all of which are located directly above the powerstation. Transmission lines will carry power from and to the powerstation from a proposed substation to be built by Commonwealth Edison Company at 145th Street and Burnham Avenue in Calumet City, Illinois.

Value of Power

The value of power generated by the Deep Tunnel Project is dependent on a variety of factors. These include the degree of dependability of the hydroelectric power as compared to alternative sources, the cost of alternative methods of providing peaking capacity, the cost of pumping energy, the manner in which the pumped-storage facility will be operated (daily operation, ready reserve, etc.), and the size of other generating units in the system, and the change in their efficiency as a result of pumped-storage operations.

The purposes of this report do not require the determination of the value of power to the degree of reliability which would justify detailed analyses of the above factors. Such studies are reserved for the feasibility-grade studies proposed as a follow-up to this report. When made, they will of necessity involve detailed coordination with Commonwealth Edison.

For purposes of this report, a range in alternative costs of providing peaking power has been selected. The lowest conceivable cost would be from gas-turbines, which have been built for costs between $60 and $70 per kilowatt. The highest conceivable justified cost would be the amounts expended by other utilities for pumped-storage installations, which have been as high as $110 per kilowatt. Even though these values do not represent true alternatives to pumped-storage peaking capacity from the Deep Tunnel Plan, the range of $60 to $110 has been selected for the preliminary evaluation purposes of this report.

Dependability of Pumped-Storage Installation

The Deep Tunnel Plan is a multi-purpose project, to serve the interests of pollution control, drainage, and power generation. The underground reservoir will serve the dual purpose of:

(a) Temporary storage of storm water prior to evacuation from the system and

(b) Storage of water released from the upper reservoir for power generation.

Studies of potential conflict between these uses are essential to establishment of design for the project. Preliminary studies of this type have been made for purposes of this report to aid in the selection of the installed capacity that would be feasible.

Relation of Climatic Condition to Power Load

The possibility of conflict between power and flood control would arise only during times when flooding occurs. This would usually rule out the winter months. although occasional rainstorms have occurred during the winter period. To aid in the judgment on the possibilities of conflict, analyses were made of the time of occurrence of major storms and the relationship between power load and climatic conditions.

The plotting in the upper left of Exhibit E-1 indicates the occurrences of runoff in the various months for the period 1871-1965. The values of runoff shown are computed rather than observed, applying the procedures outlined in Appendix A to the observed Chicago precipitation for the period. The plot indicates that three storms have occurred during the 94-year period which would have a computed runoff in excess of the storage capacity of the lower reservoir, equivalent to 2.5 inches of runoff. Thirteen storms have occurred with runoff in excess of 1.5 inches, the residual storage volume after deduction of the 40 percent of volume assumed to be available for power use. The frequency of occurrences of storms of other magnitudes is shown in the plot at the upper center of Exhibit E-1.

All of the storms exceeding 2.5 inches of runoff, and over half of the remaining storms that exceeded 1.5 inches of runoff, occurred during the period of May through September. This is the time of the year when peak power loads are anticipated in 1975. Studies were made of the relation of power load to temperatures, to allow estimates to be made of the load conditions which could be associated with the major historical rainfalls.

The plot in the lower left of Exhibit E-1 shows the relationship of peak loads in the May to September period of 1963 and 1964 to observed maximum daily temperatures. For this plotting, 1963 load values were increased by 7.38 percent, the annual growth in peak power load, to make the 1963 values comparable to those of 1964. The plot shows that a general trend exists, with load increasing significantly with temperature. This trend primarily reflects increased use of air-conditioning with increased temperature, although other factors, such as degree of sunlight and humidity, effect the relationship. It is probable that use of a temperature-humidity index would improve the relationship.

The solid line shown on the plot was used as the basis for analyses in this report. It should be noted that use of this line is conservative from the viewpoint of indicating possible conflicts, since it is an envelope of most of the plotted points rather than a line indicating the average relationship. Therefore, on a probability basis, any potential conflict in use of lower reservoir storage would have a lower probability of occurrence than indicated by use of the enveloping line.

Studies of Potential Conflict Situations

Analysis of the operational situation for historical storm situations can be carried out only by use of hourly rainfall data. At the time of this study, hourly data were available only for the 1950-1965 period. Such data undoubtedly exist in Weather Bureau files for earlier periods, and should be obtained for the feasibility studies. However, some of the largest 24-hour storms of history have occurred during the 1950-1965 period, so that analyses of these storms gives a good indication of the potential for conflict. Four major storms during this period have been studied. These include:

(a) Storm of October 3, 1954 (precipitation 5.20 in.)

(b) Storm of October 9-10, 1954 (precipitation 7.48 in.)

(c) Storm of July 12, 1957 (precipitation 6.06 in.)

(d) Storm of June 25, 1959 (precipitation 4.58 in.)

Results with Pumping Only. Exhibits E-2 to E-5 show the computation of runoff, pumping and storage requirements for the four storms if the Deep Tunnel Project is operated without normal daily pumped-storage operations. These computations show that the maximum storage requirement for the four largest storms of the 1960-1965 period would be:

Date of Storm	Storage Required (Inches)	Maximum Temperature (°F)	Peak Load Under 1964 Condition (Megawatts)
October 3, 1954	2.23	91°	5850
October 9-10, 1954	1.03	77°	4950
July 12, 1957	2.31	84°	5400
June 25, 1959	2.41	94°	5950

Three of the above storms would cause storage accumulations of greater than 1.5 inches and thus would restrict generating capability. The fourth storm caused an accumulation of only 1.03 inches and would not restrict generation.

The next step was to investigate whether the loss of pump-storage generating capability would cause any reduction in system reserve as related to the peak load for the day. It is assumed that the 1964 annual peak load was 6000 megawatts and that generating equipment was available throughout the May 1 - October 15 period as necessary to meet such a peak load, with adequate reserves. With the pumped-storage station being operated normally for pumping only, it would be a part of the reserve which should total about 15 percent of system load, or about 900 megawatts.

Under such conditions the unused generating capacity in the system (other than reserves) for the storm of July 12, 1957 would be 600 megawatts (6000 - 5400). This would be more than adequate to replace any emergency capability lost by the pumped-storage plant. The unavailability of the pumped-storage plant during that storm would cause no overall loss in system reserves.

By the same method of computation, it can be seen that, for the storms of October 3, 1954 and June 25, 1959 there would be losses in system reserves of 150 and 250 megawatts, respectively. Since this occurrence is only twice in 15 years, it should not be serious. It should be more than compensated for by the fact that outages of equivalent thermal peaking capacity would occur more frequently than with a hydroelectric unit.

The method of analysis used above is very conservative from the viewpoint of availability of the pumped-storage capacity. In the first place, the storms of the last 15 years have been more severe than earlier storms, so that the frequency is probably less than twice in 15 years. Secondly, the analysis is for a 1964 peak load situation rather than for 1975. At the later date, the difference between annual peak load and peak load for the day would be greater, perhaps doubled, so that the loss in system reserves would be less significant.

Operation with Pumped-Storage

Exhibits E-6 to E-9 show operational data for the four large storms of 1960-1965 if the power facilities are operated on a normal daily basis. Rainfall and runoff amounts are the same as shown on Exhibits E-2 to E-5. Generation is assumed to be in accordance with load curve requirements, with the full plant capacity utilized at the peak load hours and with the load reduced at other hours by the difference between the peak hour load and actual hourly load. Pumping is assumed possible according to the same schedule as in Exhibits E-2 to E-5.

The results of these operations may be summarized as follows:

Date of Storm	Storage Required (Inches)	Peak Load (Megawatts)
October 3, 1954	3.04	5850
October 9-10, 1954	1.78	4950
July 12, 1957	3.06	5400
June 25, 1959	2.41	5950

Since the pumped-storage plant is being operated normally on a daily basis, it would not be required to serve as a standby. Therefore, the full use of the 2.5 inches of storage for both generation and power production involves no restriction in planned use. On this basis the storms of October 9-10, 1954 and June 25, 1959 would cause no problem.

Exhibit E-6 shows that the lower reservoir storage would have been 2.04 inches at 11 A.M. on October 3, 1954. If operating rules were established to restrict pumped-storage operations when the lower reservoir storage had reached a specified amount (such as, for example, 1.5 inches), the situation would be the same as in the "pumping only" study. The loss of pumped-storage capability would amount to a loss in system reserves of the difference between the annual peak and the peak for that day, or 150 megawatts.

The storage requirement for the 1957 storm would be 3.06 inches, or 0.56 inches greater than available capacity if the pumped-storage plant were operated on a routine daily basis. This situation could not have been avoided unless the heavy rainstorm had been predicted since the daily generation cycle would have been completed by the time that any significant runoff occurred. Under such a circumstance, if the heavy rainstorm could not be predicted, the 0.56 inches of runoff that could not be accommodated would have caused temporary flooding.

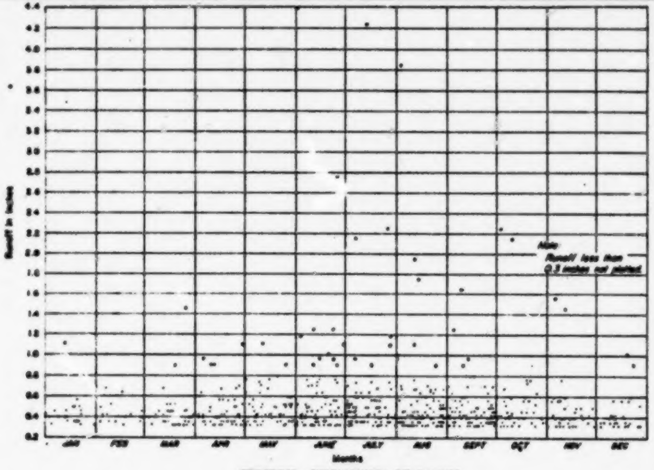
Studies of the July 1957 storm indicate that, because of its high intensity and short duration (6.06 inches in 12 hours), it is a storm which has a probable frequency in the order of once in 100 years. The chance of it occurring shortly after the completion of the generation cycle is probably even more remote than the once-in-100-years occurrence. This type of chance is not unreasonable for flood control design.

Conclusions

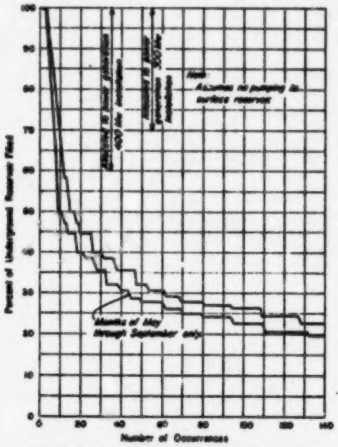
The addition of pumped-storage facilities to the Deep Tunnel Plan is highly desirable. The incremental cost of such an installation will be low, so that power facilities can be expected to contribute to the economic feasibility of the Plan. There is some potential for rare conflicts in use of the underground storage capacity, but the possibility of unavailability of the pumped-storage capacity because of such conflict is much more remote than the possibility of mechanical or electrical outage of alternative sources of peaking power.

Further studies in the feasibility stage of investigations will be required to determine the proper storage allowance, installed capacity, operating rules, and value of power.

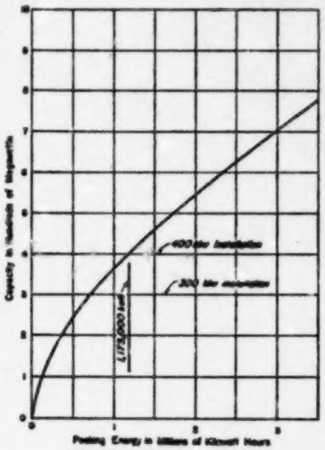
EXHIBIT E-1



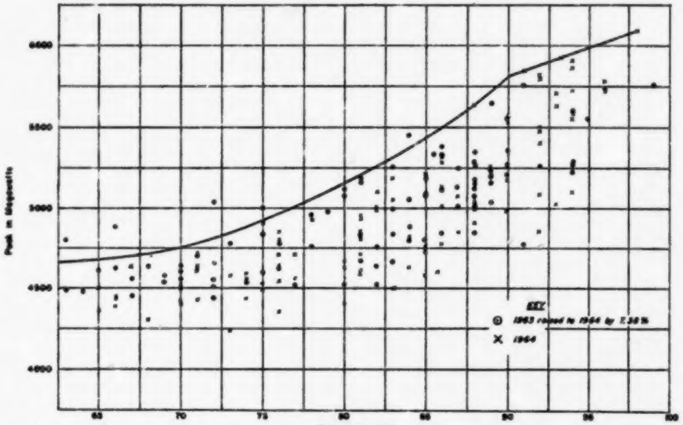
SEASONAL DISTRIBUTION OF RUNOFF



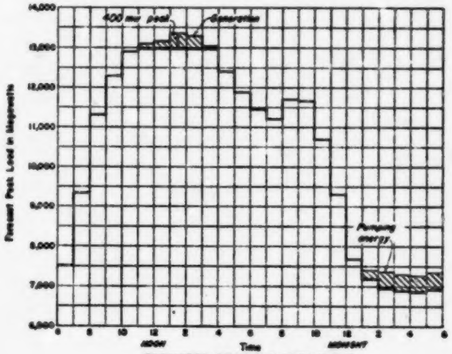
FREQUENCY OF USE OF UNDERGROUND RESERVOIR FOR FLOOD STORAGE 1871-1965



CAPACITY - ENERGY RELATIONSHIP



RELATIONSHIP OF GENERATION TO TEMPERATURE
(May through September)



ESTIMATED PEAK DAY LOAD CURVE

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

PUMPED-STORAGE
POWER RELATIONSHIPS

HARZA ENGINEERING COMPANY DWG. NO. 387 P-65
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT E-2

OPERATION WITH PUMPING ONLY

STORM OF OCTOBER 3, 1954

Maximum Temperature = 91°

			Impervious Area 1/		Pervious Area					
Hour at End of Period	Rain During Period (Inches)	Cumulative Rain (Inches)	Cumulative Supply (36% of Col. 3) (Inches)	Cumulative Run-off (Col. 4-0.06) (Inches)	Cumulative Supply (64% of Col. 3) (Inches)	Cumulative Losses 2/ (Inches)	Cumulative Run-off (Col. 6-7) (Inches)	Cumulative Total Run-off (Col.5+8) (Inches)	Cumulative Pumping 3/ (Inches)	Cumulative Storage Requirements (Inches)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
6 AM	0.54	0.54	0.19	0.13	0.35	0.35	-	0.13	-	0.13
7	0.85	1.39	0.50	0.38	0.89	0.89	-	0.38	0.25	0.13
8	1.87	3.26	1.17	0.99	2.09	1.25	0.84	1.83	0.50	1.33
9	1.49	4.75	1.71	1.47	3.03	1.52	1.51	2.98	0.75	2.23 (Maximum)
10	0.09	4.84	1.74	1.47	3.10	1.59	1.51	2.98	1.00	1.98
11	0.34	5.18	1.86	1.53	3.32	1.81	1.51	3.04	1.25	1.79
12 Noon	0.02	5.20	1.87	1.53	3.33	1.82	1.51	3.04	1.25	1.79

1/ Impervious area is 36% of total area.

2/ From graph relating infiltration and depression storage to time and intensity of precipitation.

3/ Pumping from storage at maximum rate (0.25 inches per hour) between hours of 6 PM and 11 AM providing that quantity of water is in the lower reservoir. Assume no pumping during first hour.

EXHIBIT E-3

OPERATION WITH PUMPING ONLY

STORM OF OCTOBER 9 AND 10, 1954

Maximum Temperature = 77° and 68°

			Impervious Area		Pervious Area					
Hour at End of Period	Rain During Period (Inches)	Cumulative Rain (Inches)	Cumulative Supply (36% of Col. 3) (Inches)	Cumulative Run-off (Col. 4-0.06) (Inches)	Cumulative Supply (64% of Col. 3) (Inches)	Cumulative Losses (Inches)	Cumulative Run-off (Col. 6-7) (Inches)	Cumulative Total Run-off (Col. 5+8) (Inches)	Cumulative Pumping (Inches)	Cumulative Storage Requirements (Inches)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
8 PM	0.76	0.76	0.27	0.21	0.49	0.49	-	0.21	-	0.21
9	1.24	2.00	0.72	0.60	1.28	1.00	0.28	0.88	0.25	0.63
10	0.50	2.50	0.90	0.72	1.60	1.32	0.28	1.00	0.50	0.50
11	0.35	2.85	1.03	0.79	1.82	1.54	0.28	1.07	0.75	0.32
12	0.52	3.37	1.21	0.91	2.16	1.72	0.44	1.35	1.00	0.35
1 AM	0.84	4.21	1.52	1.16	2.69	1.87	0.82	1.98	1.25	0.73
2	0.18	4.39	1.58	1.16	2.81	1.99	0.82	1.98	1.50	0.48
3	0.13	4.52	1.63	1.16	2.89	2.07	0.82	1.98	1.75	0.23
4	0.28	4.80	1.73	1.20	3.07	2.23	0.82	2.02	2.00	0.02
5	0.44	5.24	1.89	1.30	3.35	2.39	0.96	2.26	2.00	0.26
6	0.38	5.62	2.02	1.37	3.60	2.55	1.05	2.42	2.00	0.42
7	0.02	5.64	2.03	1.37	3.61	2.56	1.05	2.42	2.00	0.42
8	0.03	5.67	2.04	1.37	3.63	2.58	1.05	2.42	2.00	0.42
9	0.06	5.73	2.06	1.37	3.67	2.62	1.05	2.42	2.00	0.42
10	0.19	5.92	2.13	1.38	3.79	2.74	1.05	2.43	2.00	0.43
11	0.08	6.00	2.16	1.38	3.84	2.79	1.05	2.43	2.00	0.43
12	-	6.00	2.16	1.38	3.84	2.79	1.05	2.43	2.00	0.43
1 PM	0.01	6.01	2.16	1.38	3.85	2.80	1.05	2.43	2.00	0.43
2	0.01	6.02	2.17	1.38	3.85	2.80	1.05	2.43	2.00	0.43
3	-	6.02	2.17	1.38	3.85	2.80	1.05	2.43	2.00	0.43
4	0.03	6.05	2.18	1.38	3.87	2.82	1.05	2.43	2.00	0.43
5	0.82	6.87	2.47	1.61	4.40	2.98	1.42	3.03	2.00	1.03 (Maximum)
6	0.38	7.25	2.61	1.69	4.64	3.14	1.50	3.19	2.25	0.94
7	0.18	7.43	2.67	1.69	4.76	3.26	1.50	3.19	2.50	0.69
8	0.05	7.48	2.69	1.69	4.79	3.29	1.50	3.19	2.75	0.44

EXHIBIT E-4

OPERATION WITH PUMPING ONLY

STORM OF JULY 12, 1957

Maximum Temperature = 84°

			Impervious Area		Pervious Area					
Hour at End of Period	Rain During Period (Inches)	Cumulative Rain (Inches)	Cumulative Supply (36% of Col. 3) (Inches)	Cumulative Run-off (Col. 4-0.06) (Inches)	Cumulative Supply (64% of Col. 3) (Inches)	Cumulative Losses (Inches)	Cumulative Run-off (Col. 6-7) (Inches)	Cumulative Total Run-off (Col. 5+8) (Inches)	Cumulative Pumping (Inches)	Cumulative Storage Requirements (Inches)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
3 PM	0.32	0.32	0.12	0.06	0.20	0.20	-	0.06	-	0.06
4	0.08	0.40	0.14	0.06	0.26	0.26	-	0.06	-	0.06
5	-	0.40	-	0.06	-	0.26	-	0.06	-	0.06
6	0.02	0.42	0.15	0.06	0.27	0.27	-	0.06	-	0.06
7	2.13	2.55	0.92	0.65	1.63	0.93	0.70	1.35	0.25	1.10
8	0.37	2.92	1.05	0.72	1.87	1.17	0.70	1.42	0.50	0.92
9	0.71	3.63	1.31	0.92	2.32	1.45	0.87	1.79	0.75	1.04
10	-	3.63	-	0.92	-	1.45	0.87	1.79	1.00	0.79
11	2.02	5.65	2.03	1.58	3.62	1.65	1.98	3.56	1.25	2.31 (Maximum)
12	0.34	5.99	2.16	1.65	3.83	1.80	2.03	3.68	1.50	2.18
1 AM	0.04	6.03	2.17	1.65	3.86	1.83	2.03	3.68	1.75	1.93
2	0.03	6.06	2.18	1.65	3.88	1.85	2.03	3.68	2.00	1.68

OPERATION WITH PUMPING ONLY

STORM OF JUNE 25, 1959

Maximum Temperature = 94°

			Impervious Area		Pervious Area					
Hour at End of Period	Rain During Period (Inches)	Cumulative Rain (Inches)	Cumulative Supply (36% of Col. 3) (Inches)	Cumulative Run-off (Col. 4-0. 06) (Inches)	Cumulative Supply (64% of Col. 3) (Inches)	Cumulative Losses (Inches)	Cumulative Run-off (Col. 6-7) (Inches)	Cumulative Total Run-off (Col. 5+8) (Inches)	Cumulative Pumping (Inches)	Cumulative Storage Requirements (Inches)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
6 PM										
7										
8										
9	0.68	0.68	0.25	0.19	0.43	0.43	-	0.19	-	-
10	1.67	2.35	0.85	0.73	1.50	0.98	0.52	1.25	0.25	1.00
11	2.06	4.41	1.59	1.41	2.82	1.32	1.50	2.91	0.50	2.41 (Maximum)
12	0.17	4.58	1.65	1.41	2.93	1.43	1.50	2.91	0.75	2.16

EXHIBIT E-5

EXHIBIT E-6

NORMAL DAILY PUMPED-STORAGE OPERATION,

STORM OF OCTOBER 3, 1954

Hour at End of Period	Cumulative Flow from Generation (Inches)	Impervious Area Cumulative Run-off (Inches)	Pervious Area Cumulative Run-off (Inches)	Cumulative Generation, and Run-off (Inches)	Cumulative Pumping (Inches)	Cumulative Storage Required (Inches)
6 AM	-	0.13	-	0.13	-	0.13
7	-	0.38	-	0.38	0.25	0.13
8	-	0.99	0.84	1.83	0.50	0.13
9	-	1.47	1.51	2.98	0.75	2.23
10	-	1.47	1.51	2.98	1.00	1.98
11	-	1.53	1.51	3.04	1.00	2.04
12 Noon	0.11	1.53	1.51	3.15	1.00	2.15
1 PM	0.18	1.53	1.51	3.22	1.00	2.22
2	0.44	1.53	1.51	3.48	1.00	2.48
3	0.74	1.53	1.51	3.78	1.00	2.78
4	1.00	1.53	1.51	4.04	1.00	3.04 } Maximum
5	1.00	1.53	1.51	4.04	1.00	3.04 }
6	1.00	1.53	1.51	4.04	1.25	2.79
7	1.00	1.53	1.51	4.04	1.50	2.54
8	1.00	1.53	1.51	4.04	1.75	2.29 *

* 9 hours pumping will leave the lower reservoir empty at 6 AM the next day

EXHIBIT E-7
Page 1 of 2

NORMAL DAILY PUMPED-STORAGE OPERATION,

STORM OF OCTOBER 9 AND 10, 1954

Hour at End of Period	Cumulative Flow from Generation (Inches)	Impervious Area Cumulative Run-off (Inches)	Pervious Area Cumulative Run-off (Inches)	Cumulative Generation, and Run-off (Inches)	Cumulative Pumping (Inches)	Cumulative Storage Required (Inches)
12 Noon	0.11	-	-	0.11	-	0.11
1 PM	0.18	-	-	0.18	-	0.18
2	0.44	-	-	0.44	-	0.44
3	0.66	-	-	0.66	-	0.66
4	0.74	-	-	0.74	-	0.74
5	1.00	-	-	1.00	-	1.00
6	1.00	-	-	1.00	0.25	0.75
7	1.00	-	-	1.00	0.50	0.50
8	1.00	0.21	-	1.21	0.75	0.46
9	1.00	0.60	0.28	1.88	1.00	0.88
10	1.00	0.72	0.28	2.00	1.25	0.75
11	1.00	0.79	0.28	2.07	1.50	0.57
12	1.00	0.91	0.44	2.35	1.75	0.60
1 AM	1.00	1.16	0.82	2.98	2.00	0.98
2	1.00	1.16	0.82	2.98	2.25	0.73
3	1.00	1.16	0.82	2.98	2.50	0.48
4	1.00	1.20	0.82	3.02	2.75	0.27
5	1.00	1.30	0.96	3.26	3.00	0.26

EXHIBIT E-7
Page 2 of 2

NORMAL DAILY PUMPED-STORAGE OPERATION,

STORM OF OCTOBER 9 AND 10, 1954 (continued)

Hour at End of Period	Cumulative Flow from Generation (Inches)	Impervious Area Cumulative Run-off (Inches)	Pervious Area Cumulative Run-off (Inches)	Cumulative Generation, and Run-off (Inches)	Cumulative Pumping (Inches)	Cumulative Storage Required (Inches)
6 AM	1.00	1.37	1.05	3.42	3.25	0.17
7	1.00	1.37	1.05	3.42	3.25	0.17
8	1.00	1.37	1.05	3.42	3.25	0.17
9	1.00	1.37	1.05	3.42	3.25	0.17
10	1.00	1.38	1.05	3.43	3.25	0.18
11	1.00	1.38	1.05	3.43	3.25	0.18
12 Noon	1.11	1.38	1.05	3.54	3.25	0.29
1 PM	1.18	1.38	1.05	3.61	3.25	0.36
2	1.44	1.38	1.05	3.87	3.25	0.62
3	1.74	1.38	1.05	4.17	3.25	0.92
4	2.00	1.38	1.05	4.43	3.25	1.18
5	2.00	1.61	1.42	5.03	3.25	1.78 (Maximum)
6	2.00	1.69	1.50	5.19	3.50	1.69
7	2.00	1.69	1.50	5.19	3.75	1.44
8	2.00	1.69	1.50	5.19	4.00	1.19
9	2.00	1.69	1.50	5.19	4.25	0.94
10	2.00	1.69	1.50	5.19	4.50	0.69

EXHIBIT E-8

NORMAL DAILY PUMPED-STORAGE OPERATION,

STORM OF JULY 12, 1957

Hour at End of Period	Cumulative Flow from Generation (Inches)	Impervious Area Cumulative Run-off (Inches)	Pervious Area Cumulative Run-off (Inches)	Cumulative Generation, and Run-off (Inches)	Cumulative Pumping (Inches)	Cumulative Storage Required (Inches)
12 Noon	0.11	-	-	0.11	-	0.11
1 PM	0.18	-	-	0.18	-	0.18
2	0.44	-	-	0.44	-	0.44
3	0.74	0.06	-	0.80	-	0.80
4	1.00	0.06	-	1.06	-	1.06
5	-	0.06	-	1.06	-	1.06
6	-	0.06	-	1.06	0.25	0.81
7	-	0.65	0.70	2.35	0.50	1.85
8	-	0.72	0.70	2.42	0.75	1.67
9	-	0.92	0.87	2.79	1.00	1.79
10	-	0.92	0.87	2.79	1.25	1.54
11	-	1.58	1.98	4.56	1.50	3.06 (Maximum)
12	-	1.65	2.03	4.68	1.75	2.93
1 AM	-	1.65	2.03	4.68	2.00	2.68
2	-	1.65	2.03	4.68	2.25	2.43 *

* 8 hours pumping leaves 0.43 inches in lower reservoir at 10 AM

EXHIBIT E-9

NORMAL DAILY PUMPED-STORAGE OPERATION,

STORM OF JUNE 25, 1959

Hour at End of Period	Cumulative Flow from Generation (Inches)	Impervious Area Cumulative Run-off (Inches)	Pervious Area Cumulative Run-off (Inches)	Cumulative Generation, and Run-off (inches)	Cumulative Pumping (Inches)	Cumulative Storage Required (Inches)
12 Noon	0.11	-	-	0.11	-	0.11
1 PM	0.18	-	-	0.18	-	0.18
2	0.44	-	-	0.44	-	0.44
3	0.74	-	-	0.74	-	0.74
4	1.00	-	-	1.00	-	1.00
5	-	-	-	1.00	-	1.00
6	-	-	-	1.00	0.25	0.75
7	-	-	-	1.00	0.50	0.50
8	-	-	-	1.00	0.75	0.25
9	-	0.19	-	1.19	1.00	0.19
10	-	0.73	0.52	2.25	1.25	1.00
11	-	1.41	1.50	3.91	1.50	2.41 (Maximum)
12	-	1.41	1.50	3.91	1.75	2.16
1 AM	-	-	-	3.91	2.00	1.91
2	-	-	-	3.91	2.25	1.66

Appendix F

CONSTRUCTION METHODS AND COSTS

Introduction

A summary of estimated costs has been presented on Table V - 1 of the Main Report. This Appendix presents a discussion of the estimating procedure and the construction procedure considered in developing the estimates. The exhibits accompanying this Appendix present further details of the estimated costs for the various alternatives. Where one assumed unit price is applicable to an item on the estimates, that unit price is indicated. Many of the items, however, are summations of two or more items from the original estimates, so that a single unit price is not applicable. Numerous computations and sketches not included in this report were used in estimating quantities and developing unit prices.

Cost estimates were prepared in detail sufficient for a prefeasibility study. The project components were laid out and sized to meet their functional requirements on the basis of preliminary design computations. Quantity estimates for major items of work were made from the layout drawings and additional sketches of details as required, with frequent reference to details developed in the final design of similar components of previous projects. Estimates of unit prices represent estimated contractor's bid prices, including costs, overhead, and profit. Recent bids for similar items of work provided a guide based on experience, as did the advice of some specialty contractors. The estimates reflect cost levels prevailing in

Chicago in 1966 and take into account the higher wage rates required for underground work, two- and three-shift operation, and substantial overtime work.

Certain items of cost were estimated in much greater detail than indicated above. In particular, the underground reservoir excavation was studied carefully with regard to both cost and time because of the unusual scope of the work and its influence on project cost and schedule. Using the job cost analysis method, construction procedures and costs were studied in detail to develop realistic unit prices and rates of progress for various operations of underground excavation. Estimated contractor's costs were raised by 32.5% to allow for contractor's overhead and profit.

To the subtotal of individual amounts, on Table V-1, was added a construction contingency allowance for unforeseen difficulties in construction, possible variations in unit prices, and errors and omissions. This allowance was taken as approximately 15 percent of all items, including civil items, electrical and mechanical equipment, and land purchase. This amount is believed to be adequate considering the precedents for various project components, the available subsurface information, and the careful study of the underground excavation which accounts for the largest single item of cost.

The total direct cost, on Table IV-1, was increased by approximately 8.0 percent, to allow for indirect costs including engineering, supervision of construction, and owners' overhead.

This investigation was not intended to achieve optimum layouts or construction programs, nor to determine with precision the methods that would be used by the owner or contractors. The objective was to develop a realistic program and cost for each alternative, taking into account the designs and physical factors to the extent known at this time and the common practices developed through construction experience with similar projects. In later stages of project planning and design, it can be expected that the layouts and procedures used in the present report will be refined, thus leading to possible savings in overall construction costs.

Construction Program and Estimating Procedure

The construction details and program for each of the major project components are described in the following sections.

New Main Sewers

Construction of the new main sewers would be by conventional methods. The sewers would range from 21 to 96 inches in size and from $25 to $130 per lineal foot in cost. They would be laid somewhat deeper than ordinary sewers in order to pass under existing utilities. Generally they would be laid in open trenches. Tunneling would be used to pass under obstructions such as expressways and railways, and in areas where the sewer depth would make tunneling more economical.

An estimate was made of the required length, size, and average depth of each sewer and of the number of manholes, connections, and overflow structures. Unit prices were based on current costs for similar work. Estimated prices of tunneling in clay were based in part on recent MSD experience. Estimates for sewers in Plan 1C are shown in detail on Exhibit F-1, and in summary form, for all plans, on Exhibit F-2.

Vertical Drains

The vertical drains would range from 12 to 72 inches in diameter and from $20,000 to $166,000 each in cost. They would average about 800 feet in depth and would have steel casings. Costs were estimated on the basis of the conventional rotary or percussion well drilling methods, which would be used for the smaller shafts. The larger shafts probably could be excavated more economically by ascending or descending boring machines, similar to those which have been developed for mining operations. Estimates for vertical drains are shown in detail for Plan 1C on Exhibit F-1. Estimates for other plans were derived in a similar manner.

Underground Reservoir

While the extent and total volume of the proposed underground reservoir would be unusually large, the cross-sectional dimensions would be comparable in size with those of existing tunnels, mines, and underground powerstations.

The cost of underground excavation was estimated for two alternative methods in order to establish the most economical procedure. One method would employ conventional drilling and blasting procedures as used in excavating large underground chambers. Details of this estimate are shown on Exhibit F-3. The other method would make use of a recently-developed tunneling machine or "mole." Details of this estimate are shown on Exhibit F-4. The latter was found to be the more economical procedure for the overall plan. Using the unit prices developed in these Exhibits, estimates were made for the underground reservoirs in each plan, as shown on Exhibit F-5.

Tunneling by Mole. The mole, although a relatively new machine, has rapidly become a conventional method for tunneling in homogeneous rock. It is essentially a very large, horizontal rotary rock drill. The front face is made up of a rotating disc studded either with fixed rock bits or rolling cutters. As the disc rotates, the entire machine is pushed forward by jacks bearing on the tunnel walls. Rock cuttings, ranging up to 5 inches in size, are conveyed to the rear of the machine for transfer to hauling equipment.

Moles operating in rock similar to the Galena-Platteville dolomite have achieved advance rates of over 100 feet per day. An advance rate of 56 feet per day was used in preparing the cost estimate. This rate was based on 12.25 hours of actual operation during a three-shift day, and 4.6 feet of advance per operating hour.

Use of a 13-foot diameter mole was attempted unsuccessfully in 1962 for construction of the 79th Street water tunnel in Chicago Silurian limestone. However, this same machine was subsequently used successfully in limestone rock in Toronto after the replacement of ripper teeth, suitable for soft rock, by rolling bits, better adapted to hard rock. Based on the entire record of tunneling machines to date, there is great confidence that machines could now be provided which would be able to maintain these rates of advances. It is even likely that further progress in speed and durability of these machines can be achieved by the time of construction of this project.

Underground excavation costs for Plan 2A, the comprehensive project for 29.7 square miles, were calculated assuming the use of three mole machines. The moles and all the other construction equipment were assumed to be completely depreciated over the course of the project. Underground transport of the rock cuttings was assumed to be by mine railway to high-speed mine hoists located at three 30-foot diameter shafts. The excavated rock probably would have substantial value as a construction material since it is of high quality and would be reduced to five-inch maximum size during boring. However, to be conservative, it was assumed that rock not used in construction of the upper reservoir would be made available at the shaft head at no cost to potential users. The average unit price of rock excavation for the 38 miles of 33-foot diameter tunnels, totaling 6,300,000 cubic yards, was calculated to be $6.00 per cubic yard. The derivation of this price is shown on Exhibit F-4. This unit price also was applied to the smaller volumes of the other mole plans, on the assumption that variations in the cost of shafts and machines to be carried by each unit of excavation would be offset by the variation in the average haul distance.

The tunnel layout, which provided the required collector length and storage volume, was studied to determine the number of machines and access shafts required to meet a five-year overall construction schedule. Contractor's move-in and construction of access shafts is estimated to require one year. The three moles, progressing as described above, can complete the 38 miles of tunnel in somewhat less than four years.

Estimates of the cost of the machines and their production rates were based in part on discussions with a manufacturer of tunnel boring machines and with a tunneling contractor. Estimates were made of the cost of the operating and supporting crews and of the major items of supply, such as cutter replacement and electrical energy. Cost of underground transportation of excavated rock was based on the weighted average haul distance using trains of mine cars pulled by electric locomotives. The excavated rock would be hauled to the base of the access shaft and unloaded into hoppers from which it would be loaded into high-speed mine skips for hoisting to the ground surface. Crushing would be unnecessary because the rock excavated by the mole would be already reduced to uniformly small size.

Few internal combustion engines would be used in the tunnel in the mole scheme, as all equipment would be powered by electric motors. Ventilation would be provided as required to insure safe conditions for the men working in the tunnel. The vertical drains which intersect the tunnel route would serve as ducts for exhaust or supply of air. The major costs of the ventilation system would be supply, booster, and exhaust fans. There would be a minimum of ductwork; labor to install, relocate, and remove ducts and equipment; and electrical energy.

A common electrical distribution system would serve the moles, hauling locomotives and ventilating equipment.

The round, smooth, unshattered openings driven by the mole through sound rock should require very little support. It was estimated that 25 percent of the length of the tunnels drilled by use of the moles would require rockbolting. Their cost was determined on the basis of four bolts, each ten feet long, in every five feet of tunnel.

Excavation by Drilling and Blasting. In order to achieve greatest economy in Plan 2B, the comprehensive plan with excavation by drilling and blasting, the components of the underground reservoir were arranged so that the bulk of the required storage volume would be located near the access shafts, with the collector tunnels at minimum size required for hydraulic capacity. The collector tunnels would vary from eight to fifteen feet in diameter. The price of excavating the collector tunnels by drilling and blasting was estimated to be from $48 per cubic yard for the 8-foot diameter tunnels to $35 per cubic yard for the 15-foot diameter tunnels.

The bulk storage excavation would consist of three parallel galleries four miles long, about 32 feet wide and 75 feet high. The parallel galleries would be about 120 feet apart and interconnected to facilitate ventilation and to permit simultaneous drilling and loading operations on alternate faces. The excavation would be carried out by mining procedures using top heading followed by deep benching. The deep benching would remove about 55 feet in three or four benches and would substantially reduce the labor, drilling, and powder requirements per cubic yard. Hauling costs also would be low because large machinery could be used and the haul distance to the access shaft would be relatively short.

The unit price for the bulk storage excavation was estimated to be $5.50 per cubic yard. The derivation of this price is shown on Exhibit F-3. This price was applied to smaller quantities of gallery bulk storage excavation in the other plans on the assumption that the required volume could be provided while maintaining about the same ratio of heading to benched excavation. Plan 1C, the recommended plan, would require two galleries about 3000 feet in length.

The construction program for the storage galleries was based on work proceeding in two directions from an access shaft at the north end of the Lake Calumet, near the center of the four-mile length. A second shaft was assumed to be placed at the north end of the galleries for excavation of the collector tunnels entering at that point. A third construction access shaft would be required for the powerstation and the collector tunnels at the south end of the storage galleries. Further studies of construction procedures, at a later stage of project planning, may show that a better use of the three shafts would result in a reduction of the excavation cost, or time, or both.

From the gallery access shaft, a five-mile pilot tunnel would be driven on a north-south alignment within the heading area of the center gallery. At intervals, the pilot tunnel would be enlarged so that horizontal core borings could be made. The information obtained from the borings would be used to determine the final alignment of the storage galleries.

Three parallel gallery headings would be driven both north and south from the access shaft. The headings would be about 32 feet wide and 20 feet high, with 20 foot by 20 foot connecting tunnels at intervals of about 400 feet. Tunneling operations and ventilating systems would make use of the flexibility provided by the connected, parallel headings. The lower 55 feet of the galleries would be drilled vertically and excavated in benches. Benching could proceed fairly closely behind the headings, provided that a route be kept open for the trucks hauling rock from the headings. Loading would be done by front-end loader in the heading, and by power shovels and small dozers in the benches. Hauling in both cases would be done by off-highway trucks of about ten-cubic yard capacity.

Rock would be hauled to the base of the access shaft, where it would be processed through a crusher set below the final invert elevation of the galleries into hoppers, from which it would be loaded into mine skips for hoisting to the ground surface.

The maximum slope of the benches when used as haul roads would be 15 percent. Special procedures would be required to get excavated rock to the crusher during initial heading operations until the heading had proceeded far enough to permit construction of a haul road from crusher level to heading level.

Rock bolts would be installed at an average spacing of about five feet in the roof surface of the storage galleries and connecting tunnels, in holes drilled from the jumbo at the same time face holes were being drilled.

Tunnel ventilation during construction would be an important aspect of Plan 2B. The cost estimate is based on a preliminary design of a system to supply fresh air requirements for the estimated number of men and diesel engines and an exhaust system to remove blasting fumes. Ventilation during construction of the collector tunnels would be facilitated by the vertical drain shafts which could be used to exhaust air and blasting fumes through surface-mounted fans.

The collector tunnels would join the storage galleries at or near the crown elevation. The construction of those which enter directly into the storage galleries would begin from the top headings and would be completed before final benching operations at the intersection.

The sequence of operations controlling the construction schedule in Plan 2B would be as follows; Move-in, access shaft, pilot tunnel, exploratory drilling and evaluation, top heading to intersections with central collector tunnels, central collector tunnels, and benched excavation which had been limited by collector tunnel intersections. This sequence can be completed in five years using equipment now available. Use of moles for drilling of the collector tunnels was not considered. However, it should be noted that moles could be used for the small tunnels, even if drilling and blasting were selected for the bulk excavation, with probable savings in cost and construction time.

Access Shafts

An elevator, stair and cable shaft to the pumping-generating plant and one or more construction access shafts would be provided in each plan and would be incorporated in the permanent ventilation system.

The concrete-lined construction shafts would be approximately 30 feet in inside diameter. The depth below ground surface would be about 860 feet. Each shaft would include an elevator for personnel and equipment, one or more high-speed mine skips for hoisting excavated rock, air ducts, and space for water, electrical, compressed air and fuel conduits. A housing at the top of the shaft would contain hoisting equipment, rock hoppers for truck loading, and other services.

The shafts would be sunk by conventional mining methods, with drilling and blasting required over most of the depth. A headframe would be erected over the top of the shaft, with hoisting equipment for men, equipment, and excavated rock. Temporary support of the excavation, prior to concrete lining, would be provided by steel ring beams and lagging where the shaft was in the glacial tills. A gunite lining on wire mesh would provide temporary protection against weathering where the shaft was in rock. The unit prices for access shafts were developed from a job cost analysis of the shaft construction.

Pumping-Generating Station

Construction of the underground pumping-generating station, shown in cross section on Exhibit 9 of the Main Report, would require several major steps. The smaller underground chambers, such as those for the draft tubes, draft tube gate handling, access, and ventilation tunnels, would be excavated by conventional tunneling methods, probably using full-face headings. The machinery hall, roughly 80 feet wide, 160 feet high, and 265 feet long, and the transformer chamber would be excavated in one or more headings, plus benched excavation to the required depth. Rock bolts or temporary supports would be used as required, and each chamber roof would be permanently supported by an arched concrete lining. Excavated rock would be removed through the construction access shaft.

A reinforced concrete structure would be built within the machinery hall to contain the pumping-generating units, the spherical valves, and related equipment. Erection and installation of the major equipment would be accomplished after installation of the station bridge crane. Unit prices were obtained in part from job cost analysis and in part from experience on previous projects. Cost of equipment was based on experience with procurement of similar equipment. Pumping station and pumping-generating station estimates for the various plans are shown on Exhibits F-6, F-7, and F-8.

Surface Reservoir Intake and Penstocks

The surface reservoir intakes would be conventional tower intakes of reinforced concrete, with steel cylinder gates and trashracks. Each tower would serve two pumping-generating units, and would be the discharge point wher the unit was pumping. From each tower, a vertical shaft, about 16 feet in diameter, would be excavated and lined with concrete. At the elevation of the distributors of the pumping-generating units, the shafts would turn into horizontal tunnels and split into two penstocks serving individual units.

The shafts could be sunk from the surface, as the access shaft would be, or they could be raised from the powerstation after drilling a small pilot hole from the surface. The concrete lining would be slip-formed, except for the lower 200 feet, which would be poured around the steel penstock liner.

Unit prices for the intake excavation and concrete and steel items were based on recent experience with similar items. Unit prices for shaft excavation and concrete were based in part on the job cost analysis for the access shafts. Estimates for these items are shown on Exhibits F-6, F-7, and F-8.

Surface Reservoir and Spillway

The surface reservoir, spillway, and spillway channel are shown on Exhibit 9 of the Main Report. The reservoir for the recommended plan would require acquisition of about 145 acres of land.

The reservoir would be formed by a ring dike, with a crest length of about 8000 feet. The dike would rise about 40 feet above the foundation after stripping of organic surface material. The dike would be a zoned, rock-fill structure as shown in section on Exhibit 9, built of rock obtained from the underground reservoir. The impervious core would be made of the glacial clays which occur within the reservoir area. Excavation of core material from within the reservoir would provide additional storage volume and a graded reservoir floor. The filter material in the dike would be processed from excavated rock, and the riprap facing on the inside of the dike also could be obtained from required excavation. The dike crest width of 15 feet would include a ten-foot asphaltic concrete service road and a perimeter fence.

The spillway would be an ungated concrete overflow structure discharging through an excavated channel into the Calumet River, south of the O'Brien Lock and Dam.

Unit prices for various items of work on the surface reservoir and spillway were estimated on the basis of recent experience on similar work, with allowance for the particular conditions of the project, such as availability of materials from required excavation. Estimates for the two sizes of surface reservoir, spillway, and channel are shown on Exhibits F-9 and F-10.

Pipeline to Treatment Plant

The treatment plant conduit, designed for 150 cfs, would consist of approximately 9100 feet of welded steel pipeline, 4.0 feet in diameter. The pipe would be laid in open trench and backfilled. Where the route crosses railroads or expressways, the pipe would be jacked into place, and excavated from the inside. About three such crossings would be required. An intake and a small pumping station would be provided at the surface reservoir. Prices for the pipeline and related work were based on experience and published cost data. The estimate is shown on Exhibit F-11.

Recharge Well System

The groundwater recharge system would be constructed using conventional water well and distribution system methods. It was assumed that housing for the distribution pumps and limited storage for reclaimed water would be provided by the Calumet Sewage Treatment Plant. Prices were based on experience and on published cost data for similar work. The estimate is shown on Exhibit F-12.

Acknowledgement

Paul Weir Company, mining engineers, acted as consultants in the investigations of cost of underground excavation. They advised on construction methods and reviewed computations and results and made valuable comments and suggestions. A letter from Paul Weir Company, commenting on the underground excavation investigations is included as Exhibit F-13.

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-1

Project DEEP TUNNEL Date May, 1966 Page ____ of ____ Pages

Structure New Main Sewers & Vertical Drains Plan 1-C Estimated by ____ Checked by ____

Item No.	ITEM	Quantity	Unit Price	Amount
	New Main Sewers -21 inch dia.	14000 LF	$25.00	$ 350 000
	24 inch dia.	5000	30.00	150 000
	27 inch dia.	28500	32.00	912 000
	30 inch dia.	9500	35.00	332 500
	33 inch dia.	5900	38.00	224 200
	36 inch dia.	22600	41.00	926 600
	39 inch dia.	7800	45.00	351 000
	42 inch dia.	8600	48.00	412 800
	48 inch dia.	18400	56.00	1030 400
	54 inch dia.	14400	70.00	1008 000
	60 inch dia.	12500	75.00	937 500
	66 inch dia.	6600	80.00	528 000
	72 inch dia.	4200	85.00	357 000
	78 inch dia.	5600	95.00	532 000
	96 inch dia.	660	130.00	85 800
	Subtotal, Sewers			$8137 800
	Sewer Manholes			
	Standard, 48 inch & larger sewers	154	650.00	100 100
	Standard, 42 inch & smaller sewers	305	550.00	167 750
	Subtotal, Sewer Manholes			$267 850
	Relief & Intercepting Manholes			
	With Weir control	294	1500.00	441 000
	With Regulator control	64	5300.00	339 200
	With regulator control	13	14000.00	182 000
	Subtotal, Relief & Intercepting Manholes			$962 200
	Subtotal, New Main Sewers			$9367 850
	SUBTOTAL DIRECT COST - USE			$9400 000
	Vertical Drains, including head chambers			
	12 inch dia.	16	20000.00	$320 000
	20 inch dia.	27	37000.00	999 000
	26 inch dia.	15	47000.00	705 000
	30 inch dia.	7	61000.00	427 000
	36 inch dia.	2	68000.00	136 000
	56 inch dia.	1	105000.00	105 000
	Subtotal, Vertical Drains			$2692 000
	SUBTOTAL DIRECT COST - USE			$2700 000

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-2

Project DEEP TUNNEL **Date** May, 1966 **Page** ____ **of** ____ **Pages**

Structure New Main Sewers-All Plans **Estimated by** ____ **Checked by** ____

Item No.	ITEM	Quantity	Unit Price	Amount
	PLAN 1-A: Sewers			$ 640 000
	Sewer Manholes			20 000
	Relief & Intercepting Manholes			80 000
	Subtotal			740 000
	SUBTOTAL DIRECT COST - USE			$ 800 000
	PLAN 1-B: Sewers			640 000
	Sewer, Manholes			20 000
	Relief & Intercepting Manholes			70 000
	Subtotal			730 000
	SUBTOTAL DIRECT COST - USE			$ 800 000
	PLAN 1-C: Sewers			8 140 000
	Sewer Manholes			270 000
	Relief & Intercepting Manholes			960 000
	Subtotal			9 370 000
	SUBTOTAL DIRECT COST - USE			$ 9 400 000
	PLAN 2-A: Sewers			14 010 000
	Sewer Manholes			370 000
	Relief & Intercepting Manholes			1 530 000
	Subtotal			15 910 000
	SUBTOTAL DIRECT COST - USE			$15 900 000
	PLAN 2-B: Sewers			24 530 000
	Sewer Manholes			500 000
	Relief & Intercepting Manholes			1 790 000
	Subtotal			26 820 000
	SUBTOTAL DIRECT COST - USE			$26 800 000

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-3

Project DEEP TUNNEL Date May, 1966 Page ____ of ____ Pages

Structure Unit Price for Gallery Excavation Estimated by ____ Checked by ____

Item No.	ITEM	Quantity	Unit Price	Amount
	Estimated unit cost for heading excavation, hauled to base of shaft:			
	Drill crew & equipment			$ 3 412 000
	Powder crew and equipment			145 000
	Powder			1 112 000
	Loading crew & equipment			503 000
	Hauling crew and equipment			1 168 000
	Power, air, water lines			530 000
		1,525,000 c.y.	(4.50)	$ 6 870 000
	Contractor's unit cost, heading, use:		$4.50	
	Derive unit cost for benched or quarry excavation, hauled to base of shaft:			
	Drill crew & equipment			1 114 000
	Powder crew & equipment			145 000
	Powder			1 995 000
	Loading crew & equipment			1 388 000
	Hauling crew & equipment			3 286 000
		4,433,000 c.y.	(1.79)	$7 928 000
	Contractor's unit cost, benches, use:		$1.80	
	Access Shaft			1 100 000
	Pilot Tunnel	42,000 c.y.		1 680 000
	Heading	1,525,000 c.y.	4.50	6 862 500
	Benches	4,433,000 c.y.	1.80	7 979 400
	Rock Bolts (furnish & install)	140,000	6.00	840 000
	Crushing			1 477 000
	Hoisting			2 218 000
	Ventilation			2 600 000
	Subtotal			$24 756 900
	Contractor's Overhead & Profit		± 32.5%	8 043 100
	Estimated Price	6,000,000 c.y.	(5.47)	$32 800 000
	ESTIMATED UNIT PRICE - USE		$5.50	

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-4

Project DEEP TUNNEL Date May, 1966 Page ___ of ___ Pages

Structure Unit Price for Mole Excavation Estimated by ___ Checked by ___

Item No.	ITEM	Quantity	Unit Price	Amount
	Estimated cost of excavation by tunnel boring machine, or mole:			
	38.6 miles, 33-foot diameter;			
	Volume = 6,457,000 cubic yards			
	Access shafts (in addition to power-station access shafts)	2	1,100,000	$ 2200 000
	Moles, assembled in tunnel	3	1,600,000	4800 000
	Operation of moles			10750 000
	Hauling, to base of shafts			6600 000
	Ventilation			1800 000
	Hoisting			2600 000
	Dismantle and remove equipment			150 000
	Rock bolts			300 000
	Subtotal			29200 000
	Contractor's Overhead and Profit	32.5% ±		9500 000
	Estimated Price	6,457,000 c.y.	(5.99)	$ 38700 000
	ESTIMATED UNIT PRICE - USE		$6.00	

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-5

Project DEEP TUNNEL Date May, 1966 Page____ of ____ Pages

Structure Underground Reservoirs-All Plans Estimated by______ Checked by______

Item No.	ITEM	Quantity	Unit Price	Amount		
	PLAN 1-A					
	Mole excavation 25.2 miles	4,180,000 c.y.	6.00	$25	080	000
	Galleries	2,120,000 c.y.	5.50	11	660	000
				36	740	000
	SUBTOTAL DIRECT COST - USE			$36	800	000
	PLAN 1-B					
	Mole excavation 25.2 miles	4,180,000 c.y.	6.00	$25	080	000
	Galleries	320,000 c.y.	5.50	1	760	000
				26	840	000
	SUBTOTAL DIRECT COST - USE			$26	900	000
	PLAN 1-C					
	Mole excavation 23.4 miles	3,920,000 c.y.	6.00	$23	520	000
	Galleries	580,000 c.y.	5.50	3	190	000
				26	710	000
	SUBTOTAL DIRECT COST - USE			$26	700	000
	PLAN 2-A - SUBTOTAL DIRECT COST					
	Mole excavation - 38.6 miles	6,300,000 c.y.	6.00	$37	800	000
	PLAN 2-B					
	Gallery excavation	6,100,000 c.y.	5.50	$33	550	000
	Collector tunnels - 8 ft. dia.	34,000 c.y.	48.00	1	632	000
	9 ft. dia.	67,000 c.y.	45.50	3	048	500
	11 ft. dia.	49,000 c.y.	41.50	2	033	500
	13 ft. dia.	31,000 c.y.	38.00	1	178	000
	15 ft. dia.	19,000 c.y.	35.00		665	000
	Collector tunnel access shaft	1,100,000 x 1.325		1	457	000
		6,300,000 c.y.		43	564	000
	SUBTOTAL DIRECT COST - USE			$43	600	000

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-6

Project DEEP TUNNEL Date May, 1966 Page ____ of ____ Pages

Structure Pumping Station - Plan 1-A Estimated by ____ Checked by ____

Item No.	ITEM	Quantity	Unit Price	Amount
	Land and Land Rights (included in surface reservoir)			
	Powerstation Structures			
	Access shafts, excavation	50,000 c.y.		$ 1 325 000
	Access shafts, concrete lining	16,000 c.y.		736 000
	Underground excavation	12,000 c.y.		140 000
	Concrete Structures	2,800 c.y.		223 000
	Architectural & mechanical			342 000
	SUBTOTAL, POWERSTATION STRUCTURES			$2 766 000
	Waterways			
	Discharge line			560 000
	Suction tube			81 000
	SUBTOTAL, WATERWAYS			$ 641 000
	Pump-turbines & generator-motors	1 unit		980 000
	Accessory electrical equipment			310 000
	Miscellaneous Powerstation equipment			566 000
	Roads and Railroads			110 000
	Substation and transmission plant			750 000
	SUBTOTAL			$6 123 000
	SUBTOTAL DIRECT COST - USE			$6 100 000

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-7

Project DEEP TUNNEL Date May, 1966 Page ____ of ____ Pages

Structure Pumping-Generating Station Plans 1-B & 1-C Estimated by ____ Checked by ____

Item No.	ITEM	Quantity	Unit Price	Amount
	Land and Land Rights (included in surface reservoir)			
	Powerstation Structures			
	Access shafts, excavation			$1 325 000
	Access shafts, concrete lining			736 000
	Underground excavation	115,000 c.y.		696 000
	Concrete structures	18,900 c.y.		1 270 000
	Architectural & mechanical			780 000
	SUBTOTAL, POWERSTATION STRUCTURES			$4 807 000
	Waterways			
	Surface reservoir intake-excavation	128,000 c.y.		224 000
	Surface reservoir intake - concrete	6,300 c.y.		258 000
	Surface reservoir intake - gates & trashracks			150 000
	SUBTOTAL, SURFACE RESERVOIR INTAKE			$ 632 000
	Penstocks - shaft & tunnel exc.	14,000 c.y.		425 000
	Penstocks - concrete	5,900 c.y.		510 000
	Penstocks - steel plate	315,000 lbs.		110 000
	SUBTOTAL, PENSTOCKS			$1 045 000
	Draft tubes - tunnel exc.	4,000 c.y.		140 000
	Draft tubes - concrete	1,900 c.y.		63 000
	Draft tubes - steel plate	530,000 lbs.		186 000
	Draft tubes - trashracks	57,000 lbs.		20 000
	SUBTOTAL, DRAFT TUBES			$ 409 000
	Pump-turbines & Generator-motors	3 units		8 250 000
	Accessory electrical equipment			1 100 000
	Miscellaneous Powerstation equipment			1 079 000
	Roads and Railroads			110 000
	Substation and transmission plant			3 260 000
	SUBTOTAL			20 692 000
	SUBTOTAL DIRECT COST - USE			$20 700 000

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-8

Project DEEP TUNNEL Date May, 1966 Page ____ of ____ Pages

Structure Pumping-Generating Station Plans 2-A & 2-B Estimated by ____ Checked by ____

Item No.	ITEM	Quantity	Unit Price	Amount
	Land and Land Rights	(Included in surface reservoir)		
	Powerstation structures			
	Access shafts, excavation	50,000 c.y.		$1 325 000
	Access shafts, concrete lining	16,000 c.y.		736 000
	Underground excavation	142,000 c.y.		866 000
	Concrete structures	23,400 c.y.		1 567 000
	Architectural & mechanical			923 000
	SUBTOTAL, POWERSTATION STRUCTURES			$5 417 000
	Waterways			
	Surface reservoir intake - exc.	170,000 c.y.		297 000
	Surface reservoir intake - concrete	8,400 c.y.		343 000
	Surface reservoir intake - gates & trashracks			200 000
	SUBTOTAL, SURFACE RESERVOIR INTAKE			$ 840 000
	Penstocks - shaft & tunnel exc.	18,000 c.y.		547 000
	Penstocks - concrete	7,800 c.y.		672 000
	Penstocks - steel plate	420,000 lbs.		147 000
	SUBTOTAL, PENSTOCKS			$1 366 000
	Draft tubes - tunnel exc.	5,300 c.y.		186 000
	Draft tubes - concrete	2,500 c.y.		83 000
	Draft tubes - steel plate	705,000 lbs.		247 000
	Draft tubes - trashracks	75,000 lbs.		26 000
	SUBTOTAL, DRAFT TUBES			$ 542 000
	SUBTOTAL WATERWAYS			$2 648 000
	Pump-turbines & Generator-motors	4 units		11 000 000
	Accessory electrical equipment			1 420 000
	Miscellaneous powerstation equipment			1 191 000
	Roads and Railroads			110 000
	Substation and transmission plant			4 175 000
	SUBTOTAL			$ 26 071 000
	SUBTOTAL DIRECT COST - USE			$26 100 000

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-9

Project DEEP TUNNEL Date May, 1966 Page ______ of ______ Pages

Structure Plans 1-B & 1-C Surface Reservoir (2360 Acre-Feet) Estimated by ______ Checked by ______

Item No.	ITEM	Quantity	Unit Price	Amount
	Land and Land Rights	145 Acres	$20,000	$2,900 000
	Reservoir			
	Stripping and dredging	860,000 c.y.		640 000
	Unwatering			120 000
	Dike	1,240,000 c.y.		780 000
	Crest road and fencing	6,800 LF		64 000
	Spillway and Channel			
	Excavation	200,000 c.y.		120 000
	Concrete	1,300 c.y.		72 000
	Bridges			50 000
	SUBTOTAL			$4 746 000
	SUBTOTAL DIRECT COST - USE			$4 700 000

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-10

Project DEEP TUNNEL Plans 2-A & 2-B Date May, 1966 Page ______ of ______ Pages

Structure Surface Reservoir 3320 Acre-Feet Estimated by ______ Checked by ______

Item No.	ITEM	Quantity	Unit Price	Amount
	Land and Land Rights	176 Acres	20,000	$3 520 000
	Reservoir			
	Stripping and dredging	1,020,000 c.y.		750 000
	Unwatering			120 000
	Dike	1,470,000 c.y.		922 000
	Crest road and fencing	8,000 LF		74 000
	Spillway and Channel			
	Excavation	236,000 c.y.		142 000
	Concrete	1,600 c.y.		88 000
	Bridges			50 000
	SUBTOTAL			$5 666 000
	SUBTOTAL DIRECT COST - USE			$5 700 000

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-11

Project CAVERN RESERVOIR Date May, 1966 Page ____ of ____ Pages

Structure Pipeline to Treatment Plant Estimated by ____ Checked by ____

Item No.	ITEM	Quantity	Unit Price	Amount		
	(150 cfs)					
	4-ft. diameter steel conduit, installed in open trench and backfilled. 500 LF ± jacked into place under highways	9,100 LF	80.00	$	728	000
	Pump house and miscellaneous		LS		22	000
	Pumps and motors		LS		100	000
	SUBTOTAL				850	000
	SUBTOTAL DIRECT COST-USE			$	800	000

ESTIMATE

HARZA ENGINEERING COMPANY
BAUER ENGINEERING, INC

EXHIBIT F-12

Project DEEP TUNNEL Date May, 1966 Page ____ of ____ Pages

Structure Recharge Water Distribution Estimated by ____ Checked by ____

Item No.	ITEM	Quantity	Unit Price	Amount
	Recharge wells	17	$20,000	$ 340 000
	Rehabilitation pumps	2	10,000	20 000
	Distribution pumps	1	15,000	15 000
	Water Distribution			
	RR & Hwy Crossings & Easements	5,000 LF	50.00	250 000
	6 inch dia. c.l. water main	22,000 LF	13.00	* 286 000
	10 inch dia c.l. water main	6,000 LF	18.00	* 108 000
	12 inch dia. steel water main	33,000 LF	19.00	* 627 000
	16 inch dia. steel water main	30,000 LF	23.00	* 690 000
	Well head structures	17	10,000	170 000
	Well head facilities, meter, siphon breaker, valves, instrumentation	17	3,000	51 000
	Mobile crane or service rig	1	50,000	50 000
	Chlorinator, analyzer, other instruments at plant	1 set	20,000	20 000
	SUBTOTAL			2 627 000
	SUBTOTAL DIRECT COST - USE			$2 600 000

* Includes 4.00/LF for pavement breaking and replacement

EXHIBIT F-13

CABLE ADDRESS "WEIRCO"

PAUL WEIR COMPANY
INCORPORATED
MINING ENGINEERS AND GEOLOGISTS
(312) FINANCIAL 6-0275
20 NORTH WACKER DRIVE
CHICAGO, ILLINOIS 60606

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DONALD H. DOWLIN
DAVID J. KACHIK

May 13, 1966

Mr. Richard D. Harza
Harza Engineering Company
400 West Madison Avenue
Chicago, Illinois 60606

Dear Mr. Harza:

The Paul Weir Company has assisted the Harza Engineering Company in the pre-feasibility investigation of the Deep Tunnel Plan. For the purpose of developing cost estimates, we have made suggestions as to suitable equipment and methods of excavating and constructing the tunnels. We have reviewed the Harza estimates of cost for tunnel drivage and give you herewith our opinions as to their adequacy.

Geologic investigations by Harza have been limited to careful study of information presently available. The Harza geologists have concluded that the rock would be essentially self-supporting and that the completed tunnels would require no concrete lining and only a minimum of roof support. Our comments as to costs are based on the above conclusions of Harza.

The cost estimates cover two basic methods of tunnel drivage, the first incorporating large tunnel-boring machines and the second, conventional drivage by drilling, blasting and loading out. Further study will be required to confirm the feasibility of tunnel-boring.

We believe that the Harza cost estimates have been carefully prepared and represent capable engineering. The unit costs for tunnel-boring and for conventional drivage are, in our opinion, adequate on the basis of the conclusions of the Harza geologists.

Respectfully submitted,

PAUL WEIR COMPANY

J. P. Weir

J. P. Weir

JPW/if

Appendix G

BENEFITS FROM FLOOD AND POLLUTION CONTROL

Introduction

The Deep Tunnel Plan will provide substantial benefits, both in the Lake Calumet Area and downstream along the Illinois River Waterway. The most significant benefits will be derived from the flood control, pollution abatement, and pumped-storage features of the plan. Other benefits also will accrue, many of which cannot be assigned a monetary value. Tangible benefits have been estimated in monetary terms in this study where possible, while intangible benefits are discussed in qualitative terms.

To assess tangible benefits, an examination has been made of available reports, together with elementary field and office studies. Tangible benefits are generally of the nature of an elimination of a damage hazard, an increase in value of property, or a reduction of a cost of providing a service. Examples of tangible benefits are:

(a) Reduction of the risk of flood damage;

(b) Increase in property value and potential use with elimination of flooding hazard; and

(c) Lowered costs of drainage improvements.

The estimated annual value of tangible benefits was reduced to equivalent present worth for each item. This permits a direct comparison of the estimated benefits to the costs of the proposed plan.

The Deep Tunnel Plan will produce many benefits which cannot be assessed in monetary terms either because monetary values are inappropriate or because analytical methods require data which are not presently available or are beyond the scope of this study. For example, the plan will substantially reduce the inconvenience and traffic disruption associated with construction of conventional sewers in streets. This safety harzard and disturbance to business and service activities would have a real and substantial value, but it has not been possible to calculate the value in dollar terms.

More exhaustive economic studies may permit the determination of monetary values for benefits now classed as intangible. Such studies should be possible during the next phase of investigations.

Flood Protection and Drainage

Basement Flooding

The Calumet Area is subject to extensive basement flooding, during even relatively minor storms. Much of this area has not been provided with sewers that afford the same level of protection found in other portions of the City. In part of the area, existing main sewers do not afford adequate relief for lateral sewers, which then cause flow to back up into basements. Property is damaged in many basements which are fully utilized, and utilization of other basements is restricted by the flooding hazard. Various protection devices are used by some property owners to prevent sewer flow from entering the basement; however, the volume of water excluded from the protected basements must be accommodated - presumably in other basements.

An investigation was conducted in the Calumet Area to establish the magnitude of this basement flooding damage. The investigation was carried out in three phases. First, approximate basement elevations were determined by a field survey in three sample areas. Second, office studies were made to estimate the number of basements that would be flooded, the depth and frequency of flooding, and the resulting damages for the sample areas. Third, a direct-mail survey was conducted and evaluated for about one-half of the property owners in the sample areas.

The first two hydrologic phases of this study were completed in the manner described in detail in Appendix A. In summary, the findings related the estimated basement depth to the hydraulic gradient in the lateral sewer for storms of various frequencies and durations. The estimated hydraulic gradient was modified for flow diverted to and stored temporarily in basements. Average stage (depth-of-flooding) frequency curves were derived for each of the lateral sewers in the study areas.

Sample Areas. The three sample areas included about 1,100 houses of the estimated 20,800 houses with basements in the service area of the recommended plan. Three general ranges of housing cost were represented, based on U. S. Census of Housing cost data.

The first area is located between 103rd and 105th Streets and includes Avenues J, L, M, N, and Ewing. The houses are predominantly in the lower value range, having an average valuation of about $12,000. The second area was selected between 87th and 90th Streets, including Euclid, Bennett, Constance, Cregier, and Ridgeland Avenues. These houses are predominantly of higher value, having an average value of about $22,000. The third area is located from 105th to 107th Streets and includes Calumet, Forest, Prairie, Indiana, and Edgebrook Avenues. Houses in this area are predominantly middle-value houses with an average value of about $18,000.

Office Analysis of Damages. Basement flooding damages are related to depth of water entering the basement and frequency of flood occurrence. Damages for the area were calculated by multiplying the probability of each stage by the corresponding damage, and adding up these expected values.

Damage-stage curves were made for each of the 14 lateral sewers in the sample areas, using the Residence Flood Damage Table" of the Soil Conservation Service, [1] with modifications based on flood damage surveys conducted previously in Morton Grove, Illinois, by Bauer Engineering, Inc. The method relates basement flooding damage to house valuation, with the assumption that flood damage benefits include all flood damage repair costs. The resulting damage-stage relationships are summarized in Table G-1.

Table G-1

BASEMENT FLOODING DAMAGE

Depth	Low Value Houses ($12,000)	Middle Value Houses ($18,000)	High Value Houses ($22,000)
0.5 ft.	$ 55	$ 90	$110
1.0	110	180	220
1.5	165	270	330
2.0	220	360	440
2.5	275	450	550
3.0	330	540	660
3.5	385	620	760
4.0	440	720	870

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[1] "Residence Flood Damage Table", Soil Conservation Service. Lincoln, Nebraska, 1964.

The individual damage-stage curves were combined with the stage-frequency curves to provide a damage-frequency curve for each lateral. The area beneath each curve is equal to annual expected flood damages for the lateral under consideration. Residual damages after the completion of the recommended plan were then calculated, using the same method, and subtracted from the present expected damages to obtain net damage reduction attributable to the Plan.

The average annual damage was calculated for the three sample areas and found to be $43,000. This sample-area estimate was then expanded to cover the area to be served by the recommended plan. In expanding the results, the number of houses in the low, medium, and high value categories were taken into account. The expected damages of the low value sample area were used to estimate the expected damage of all low value houses in the study area, and similarly for the middle and high value houses. On this basis the recommended plan would result in estimated net annual damage prevention of about $700,000, or a capital saving of $11 million when expressed as present worth at six percent interest over a 50-year period.

Sample Survey of Damages. Survey questionnaires were sent to 500 residents in the sample areas, representing about one-half of the houses in these areas. Occupants were questioned about flooding occurrences and depths using the sample questionnaire shown on Exhibit G-11, to check the office analysis reported above. Over 30 percent of the residents replied.

The findings of this preliminary sample survey indicate that the results from the office analysis were substantially correct for two of the three sample areas. The third area had less basement flooding than was predicted in the office analysis. This was due to two reasons not considered in the office analysis:

(a) the houses drained to a separate sewer in the alley;

(b) the yards around the houses were lower than the street pavements and, as a result, provided a large volume of temporary storage.

Overall, the office analysis appears to have established representative values for the upper range of benefits that might be attributed to flood damage prevention.

Enhancement of Basements. The sample survey included questions to determine the number of homes which utilize basements for recreation rooms. These findings were used to estimate the potential enhancement to existing property if the flooding hazard is eliminated.

The survey returns indicated that 30 percent of all residentes in the area could be considered to desire recreation rooms if flooding is eliminated. There would be an intangible value to this enhancement since the owner would benefit from an increased intensity of basement use. On the aggregate, this intangible benefit could be very significant.

Enhancement of Undeveloped Land

There are 2,700 acres of undeveloped land included in the recommended plan area. The value of this undeveloped land would be enhanced by the reduction of the flooding hazard and by the reduction of construction costs for any future sewer construction.

No attempt has been made to estimate the magnitude of this benefit. However, the value should be very substantial, since the area has a relatively large quantity of open land, extensive transportation facilities, and has been zoned for industrial development.

Other Flood Prevention Benefits

The Plan would provide other benefits which have not been assessed in monetary terms. These benefits include the elimination of street underpass flooding, retirement of storm-water pumping stations, reduction of down-stream flooding, and improvements to waterway navigation.

Data were studied for 12 underpasses on the Chicago Transit Authority routes, and these alone were subject to about 40 hours loss of use annually. Other underpasses are probably affected to a greater extent, since they might not have the high level of maintenance and supervision available on CTA Routes.

Many storm-water pumping stations for underpasses and other facilities could be retired by the recommended plan. The elimination of annual operating and maintenance costs would represent a benefit to the Plan.

The recommended Deep Tunnel Plan would reduce storm flow contribution to the Calumet River by about 5000 cfs for the area. This would result in lower peak discharges and flooding damages downstream along the waterway. Some minor benefits might accrue to the reduced backwater effects along the Calumet River System.

Storage of the storm water would reduce interference to navigation operations during flood peaks. The controlled release of this water might permit more efficient use of Lake water diversions.

Pollution Abatement

Description of the Waterway Pollution Problem

Gross pollution exists in the waters of the Upper Illinois River System, as shown by both casual observation and scientific studies using every common indicator of pollution. Rubbish, garbage, oil slicks, and floating sewage solids, along with gas bubbles from decomposition of sludge deposits, may be observed along most of the Waterway above Lockport. Field investigations and laboratory tests have shown consistently high levels of solids, nutrients, and resulting plant growths, detergents, chlorides, toxic metals, oxygen demand, and the coliform bacteria and fecal streptococci, which are indicators of human wastes.

Historical Background

The Illinois River Waterway shown on Exhibit G-2, has undergone spectacular change in the past 100 years. A recent report (Starrett, et al[3/]) recounts this change which has been documented by an unprecedented number of scientific studies. The report concludes that the waterway has deteriorated due to the activities of man. Population in the watershed has increased from about one-half million people in 1850 to over 8.5 million people in 1960.

The river environment was first disturbed by farming, although little concern was expressed about the early changes. Diversion of Lake Michigan water and Chicago sewage began with spillage at La Salle from the Illinois-Michigan Canal in 1871, and was substantially

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[3/] "Man's Effect Upon the Fish and Wildlife of the Illinois River," by Dr. William C. Starrett, Frank C. Bellrose, and Harlow B. Mills. Illinois Natural History Survey Biological Notes, No. 57. 1966.

increased in 1900 by the opening of the Sanitary and Ship Canal. The increased diversion (coincidentally with the introduction of carp to the river) initially had a beneficial effect on commercial fishing (Exhibit G-3). However, the regime was severely damaged in the next few years by drainage for farming of about one-half (200,000 acres) of the flood plain lakes and by the accelerated growth of Chicago area waste output. By 1917 fish were not able to live above Utica because of the putrescent state of the river. Treatment improvements were made by the Metropolitan Sanitary District in the 1930's; and, although diversion was reduced substantially during this and subsequent periods, river conditions were significantly improved. Further improvements were made in recent years, but the waste load growth outstripped these remedial measures.

Several parameters demonstrate the extent of this deterioration. The turbidity of river water has increased two to three fold in the past 20 years, due to the extensive development in the watershed of row crop agriculture, barge-tow operations, and sewage pollution. Dissolved oxygen has declined to critical levels throughout the waterway. Dissolved oxygen determinations were apparently not made by early researchers, but the river once supported 18 species of fish no longer able to survive in these waters.

Other parameters could be cited, but the Starrett study concluded that the effects of these measurable indices is best assessed by changes on biologic life which also reflect the net effect of many unobserved constituents.

Tiny fingernail clams were once numerous, but they are now absent in the upper three-fourths of the river system. With the disappearance of this food, drastic reductions occurred in the populations of the blue-bill duck and certain fish. Aquatic vegetation also has disappeared and caused corresponding losses to vegetation-feeding ducks and some fish. Not all of these effects are indirect. Fishing was once so important at Havana, Illinois, that special trains were operated to bring anglers from Springfield and to carry live fish to the New York Market. The catch from the entire Beardstown to Hennepin reach of the river is now less than 1 million pounds annually. The present level of pollution along the river is indicated by the occurrence of the pollution-deformed (knothead) carp fish, as shown on Exhibit G-4.

Not all of this decline can be attributed to stream pollution. Biologic and climatological cycles, at times, have had profound effects on the flora and fauna of the river. However, the present depletion of waterway aquatic life must, to a large extent, have been caused by cumulative adverse effects of pollution. Sport fish do not exist in the upper river, and populations of these fish recently have declined sharply in the middle reaches[3/].

Pollution Attributable to Sewer Overflows

Storm overflow from combined sewers in the service area of the Sanitary District causes a significant proportion of the waterway pollution. Avalable data (U. S. Public Health Service[4/] [5/]) indicate that the average annual overflow includes three to five percent of the

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[3/] Ibid.

[4/] "Pollutional Effects of Stormwater and Overflows from Combined Sewer Systems," U.S. Public Health Service Publication No. 1246. 1964.

[5/] "Interim Report - Great Lakes-Illinois River Basin Project, "U. S. Public Health Service. 1961.

total annual volume of untreated domestic sewage. Up to 90 percent of the raw sewage in the sewer system can be spilled during some storms. Four to six overflows occur monthly, on the average.

Overflows from combined sewers are estimated to account for about 16 percent of the present average annual BOD (Biochemical Oxygen Demand) loading in the waterway, or about 24 percent of loading remaining after certain recommended improvements to other pollution sources. The drainage area encompassed in the recommended plan accounts for 1.8 percent and 2.7 percent, respectively. All of these percentages are based on total loadings in the waterway above Lockport. The Calumet area would account for a larger percentage of loading in the Calumet River and Calumet Sag Channel.

It may be assumed that storm overflows contribute a smaller proportion of coliform bacteria, and a much larger proportion of solids and floating objects. Loadings are very high during the initial period of a storm and drop off rapidly thereafter.

Quantitative evaluations of overflows are based on the Roscoe Street sewer study (U. S. Public Health Service[6/]). Overflows in the service area of the Sanitary District are estimated to cause an average annual total of 13.9 million pounds of five-day BOD (20.4 million pounds ultimate BOD) in the waterway. Overflows in the recommended plan drainage area (21.0 square miles) are estimated to cause 1.5 million pounds of five-day BOD per year (1.7 million pounds ultimate BOD).

[6/] "Report on the Illinois River System, Great Lakes - Illinois River Basin Project". U. S. Public Health Service. 1963.

Present Abatement Programs

Programs have been initiated recently for the abatement of both industrial and municipal wastes now entering the waterway system and the lake. The Upper Illinois River Waterway (Exhibit 2) receives the following population equivalent in wastes above the confluence of the Sanitary and Ship Canal with the Des Plaines River[6/].

Source	Load*
MSD Plants	770,000
Industrial	291,000
Municipal-without Secondary	48,000
with Secondary	118,000
Combined Sewer Overflows	175,000
	1,402,000

* expressed as population equivalent

The three waste treatment plants operated by the Sanitary District currently perform at reduction efficiencies of 90 percent on the influent load. A continuing program of testing and evaluation is conducted to further reduce the waste load in this area. Current plans include chlorination of the Calumet Plant effluent and aeration of a test reach of the Canal. Industrial loads might be reduced in the future by District implementation of user-service charges. These charges have been imposed in proportion to the waste load discharged by the industry. Where such charges have been assessed, they generally have resulted in industry action to reduce the waste loadings discharged to the system.

[6/] "Report on the Illinois River System, Great Lakes - Illinois River Basin Project". U. S. Public Health Service. 1963.

Municipal and industrial waste loads from plants outside of the Sanitary District should be reduced in the future, as the owners have been ordered to provide secondary treatment.

Alternatives for Overflow Pollution Abatement

Several alternative methods have been proposed for the elimination of the pollution load caused by overflows from combined sanitary and storm water sewers. These include construction of overflow holding and treatment facilities, modifications to combined sewers to provide separate systems, chlorination of overflows and separation at the individual household with conveyance of sanitary wastes in a pressure pipe within the existing sewer.

Overflow pollution could be eliminated by storage and treatment of combined sewer overflows. This method would provide a complete solution to the problem but depends on availability of economical, safe storage sites. Storage sites are not readily available in Chicago, or in the other major cities having combined sewer systems. The river banks where overflows occur are congested, and the storage volumes required are very large. Approximately 1,100 acre-feet of storage is required to hold one inch of runoff from the 21.0 square mile study area.

It appears that provision of sufficient treatment plant capacity to accommodate overflows would be even more expensive. Extremely large hydraulic capacity would be required, and treatment processes are difficult to control under intermittant conditions of high flow and low organic loadings. McKee[7/] has analyzed the relationship of

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[7/] "Loss of Sanitary Sewage through Storm Water Overflows," by J. E. McKee. Journal Boston Society of Engineers. 1947.

interceptor capacity to the amount of sanitary sewage lost and the frequency of overflows. He estimated that 2.6 percent of sewage is lost where interceptors are designed for twice the dry-weather flow (DWF). For a design ratio of five times DWF, 1.2 percent would be lost. For a design ratio of 50 times DWF, 0.1 percent would be lost, but an overflow would still occur about once a month. Treatment plant capacity would have to be increased to handle the intercepted flows, to prevent by-passing at the plant. Apparently this is not an economical solution. It is still possible, however, that a system of larger interceptors, storage tanks, and enlarged treatment plants would be more economical than the alternative of complete separation.

Separation of storm water from sanitary sewage has been the goal of most investigators. In addition to great cost, this would produce great inconvenience during construction, with resulting indirect costs. Separation would greatly reduce waterway pollution. It should be noted, however, that storm water from urban runoff frequently contains high loadings of BOD and coliform, in addition to salts, solids, and trash. Separation would generally eliminate sewer backups in basements but would have little effect on street flooding. Cost of separation in Chicago has been estimated to average about $17,000 per acre of drainage area ($230 million for the 13,500-acre Calumet Area). This is several times as large as the estimated cost of the Deep Tunnel Plan.

Professor Fair and others have proposed a method of separation which is now being studied by an ASCE Task Group[8/]. With this

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[8/] "Separation of Sanitary Sewage from Combined Systems of Sewerage," by G. M. Fair. U.S.P.H.S. Demonstration Grant. American Society of Civil Engineers. 1965.

system, domestic sewage from each building would be processed through a grinder and pumped through a small plastic pipe, placed inside the existing combined sewer, to an interceptor for sanitary sewage. The existing combined sewers would be used for storm water. This may prove to be an economical alternative to conventional separation, but it may have some drawbacks. First, the concept is not merely a construction program, to be accomplished as funds are available, but a major sales program which would require broad public support and enabling legislation. Second, the processing-pumping unit in each house might be hard to sell to the public. Third, the units would be dependent on electric power, with no by-pass possible.

Some study has been made of the detention and partial treatment of combined sewer overflows. This method differs from the storage and treatment method primarily in that smaller storage volume and less extensive treatment are provided. These might be limited to chlorination, with settlement prevented. Or they could include combinations of course screening, skimming, commutation, and limited settling. Chlorination tanks of this type are being provided in New York City, for the single purpose of disinfection to protect beaches and shell-fish beds, at a cost on the order of $1100 per acre of drainage area. The extent of BOD reduction has not been determined.

Chlorination of the waterway has been suggested as a feasible method of coliform control and BOD reduction. It could be used in addition to other methods; and, if undertaken, it would probably be in conjunction with chlorination of treatment plant effluents. No cost data are available for this alternative.

Other pollution abatement measures have been proposed, but many are not feasible alternatives for the treatment of overflow wastes. Streamflow aeration has been proposed for low-flow conditions when very little of the oxygen demand could be attributed to overflow pollution. Dilution is similarly a consideration which is not appropriate to the greater portion of the overflow load.

Pollution Abatement Benefits

It is extremely difficult to determine pollution abatement benefits. Existing levels of pollution load are not firmly established, and the water quality requirements of present and potential users is not well defined. Thus, it is virtually impossible to estimate the monetary value that pollution abatement might accrue.

The present polluted condition of the waterway does incur a public cost due to the loss of utility, increased treatment cost, and impairment of the waterway. Several aspects of waterway utilization would benefit from the effects of pollution abatement. A portion of these benefits would accrue to the elimination of overflows if the other abatement activities proceed concurrently. The following items were considered:

Waterway Aesthetics. The Upper Illinois River System is, generally, a blighted waterway. Waterways in metropolitan areas, while seldom "clean streams," frequently are scenic assets. In many cities, popular boat tours are conducted along the canals or rivers in industrial areas, and water-front areas are desired for housing. However, pollution has limited these endeavors along the upper Illinois River System.

Much of the property along the waterway is industrial in character and seemingly is not adversely affected by waterway appearance. There are extensive areas, however, where the adjoining uses are primarily residential, commercial, and recreational. These include much of the North Shore Channel and North Branch below the Northside Sewage Treatment Plant; the Chicago River through the "Loop" area; the recreational areas, present and potential, south of Willow Springs; and much of the Calumet Sag Channel and Calumet River areas. Improvement in these areas would lead to prompt enhancement of the utilization and property values. Similar benefits would be expected along the Des Plaines and Illinois Rivers below Lockport, especially in the City of Joliet.

Bramer[9] has assigned a value to this enhancement function of the Ohio River on the basis that one-percent of the population could benefit from river-viewing residences and would pay 10 percent higher rents (or equivalent purchase price) for this benefit. There is no basis for utilization of this procedure in the Chicagoland area.

Municipal and Industrial Water Supply. The existing pollution restricts the use of Illinois River water as a source of municipal supply and probably adds to water treatment costs. This problem is not as critical as it might be, because other sources have been available up to now. Peoria and Grafton are the only municipalities now obtaining water from the river. Total pumpage is about 15 mgd., and benefits from pollution abatement would probably be insignificant.

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[9] "The Economic Aspects of the Water Pollution Abatement Program in the Ohio River Valley," by Henry C. Bramer. Doctorat Thesis. University of Pittsburgh. 1960.

Costs probably are increased for most industrial uses by the presence of abrasive or corrosive materials. Certain industries may be unable to use the water or may incur increased expense due to the presence of dissolved minerals harmful to process or products. Total industrial pumpage, exceeds 1300 mgd, with nearly 1000 mgd in the Upper Waterway. Thermal power cooling use is about 7000 mgd additional. There is little data available concerning the change in Waterway mineral quality with pollution or of the related extent and cost of water treatment for various industries. However, substantial benefits probably would be realized from the reduction in industrial water treatment costs resulting from pollution abatement.

Fish and Wild Life. Fish and wildlife populations have been adversely affected by polluted water. The effects are felt in the loss to the general environment, the limiting of recreation opportunities, and the reduction of income from both commercial and recreational fishing.

There is almost no fishing in the waterway upstream from Morris except in the Kankakee and Des Plaines Rivers. Fishing in the main stem begins only below Starved Rock Dam. Some measure of this loss may be indicated by the reported[10] annual sport fishing value ($71,000 in 1954) determined for a small central Illinois lake distant from any population center. Substantially greater benefits would accrue to fishing facilities nearer to the metropolitan area.

Waterfowl populations in the Illinois-River Flyway have declined in recent years. The apparent cause is the virtual disappearance, in the flood plain lakes, of the molluscs and aquatic plants on which the ducks feed. Pollution and siltation seem to have accelerated the degradation of these lakes.

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[10] "Fishery Values of a Restored Illinois River Bottomland Lake", by Starrett, Dr. William C. Illinois Natural History Survey Reprint. 1957.

Recreation. Pollution virtually prohibits recreational use of the Upper-Illinois River Waterway. Although swimming and water-skiing have been reported, few care to take the risk. Pleasure boat travel consists mostly of transit between Lake Michigan and the Illinois River. This lack of use of the Waterway for recreation may exist, in part, because of the proximity of the lake and other more attractive waters for boating. Potential boating activity is not being fully developed or is being diverted from the waterway to more remote areas because of the unpleasant appearance and dangerous condition of the water. The same restrictions limit other water-oriented recreation such as picnicking.

Navigation. Navigation uses of the waterway are not greatly affected by pollution, although solids and sludge may add to sedimentation, and corrosion of barges and towboats may be a problem. This would be reflected in higher costs for maintenance dredging and corrosion protection. Navigation uses of the waterway contribute to high turbidities by the agitation of bottom sediments by the tows.

Irrigation. There is a small amount of irrigation in the Illinois River Basin. Over half of the acreage is irrigated from wells and most of the remainder from tributary streams. Little irrigation water is taken from the Illinois River. There are areas suitable for expansion of irrigation along the river south of Peoria. If such expansion included crops other than fiber or field crops, the bacterial contamination of the water would become important and perhaps limiting consideration.

Power. Hydroelectric power is of minor importance along the Illinois waterway, and it is unlikely that pollution causes significant costs to this use.

Public Health. The waterway is a public health hazard in its present condition. Few people come in contact with the water, so the danger of epidemic may be slight. Not all people are aware of the hazard, however, and the potential danger and restriction to use persists, in diluted form, for many miles downstream.

Other Plan Benefits

The Deep Tunnel Plan will result in other benefits in addition to flood control and pollution abatement.

Benefits from the pumped-storage power installation are discussed in detail in Appendix E. Limited benefits will result from regional improvements in ground-water levels resulting from aquifer recharge operations, as discussed in Appendix D.

About 4.5 million cubic yards of high grade crushed stone will be excavated from the proposed tunnels. These studies have not assumed any salable value for this stone, because the high production rate from tunneling would saturate the commercial market. Sales of stone now run about two million tons annually, and production of tunneling stone would run about 1.5 million tons annually.

However, if construction should be planned for specific projects which require substantial quantities of high-grade crushed stone, the excavated rock could have substantial value. The proposed Lake Calumet or Lake Michigan port fill operations could require substantial quantities of stone. Commercial prices now range from $4 to $6 per ton for crushed stone in the Chicago area. It was assumed, in the preparation of the construction cost estimates, that the excavated rock would have sufficient value that it would at least be removed from the shaft head at no cost to the project.

In addition to the benefits obtained by eliminating the overflows, the Plan would aid waste treatment plant operations substantially by reducing the hydraulic surge and disruption to biologic processes which accompanies storm water inflow. Thus, the waste load in the overflow would be treated and normal treatment plan efficiencies would be improved.

EXHIBIT G-1

Questionnaire On Basement Flooding

1. Name ______________________________
2. Address ______________________________
3. Length of Residence ______________________
4. Usages of basement: ______________________
   ____
   (family or recreation room, laundry heating plant, storeroom, etc.)
5. Please check the type of drainage system connected to basement floor drain.

   Open to sewer, without valve or pump ________
   Removable plug or pipe riser ________
   With check valve ________
   Connected to sump pump ________

6. Has basement flooding ever occurred? ________
   If yes, please answer the following:

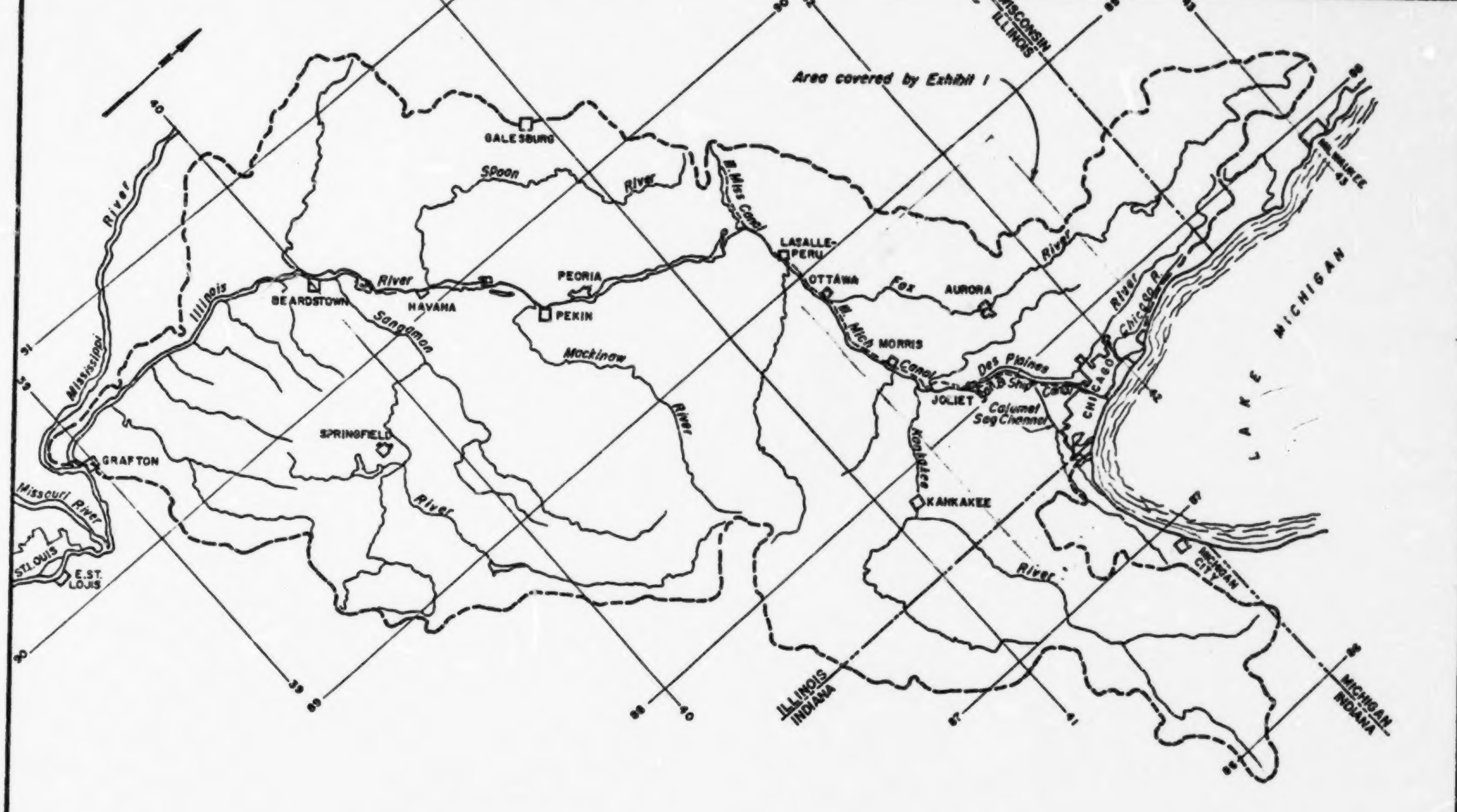
   (a) About how many times ________
   (b) Approximate depth of water in basement during most severe occurrence ________
   (c) Have you changed drainage facilities since time of flooding ________

   If yes, what type

   Plug or pipe riser ________
   Check Valve ________
   Sump Pump ________
   Other ________

7. Additional comments:

EXHIBIT G-2

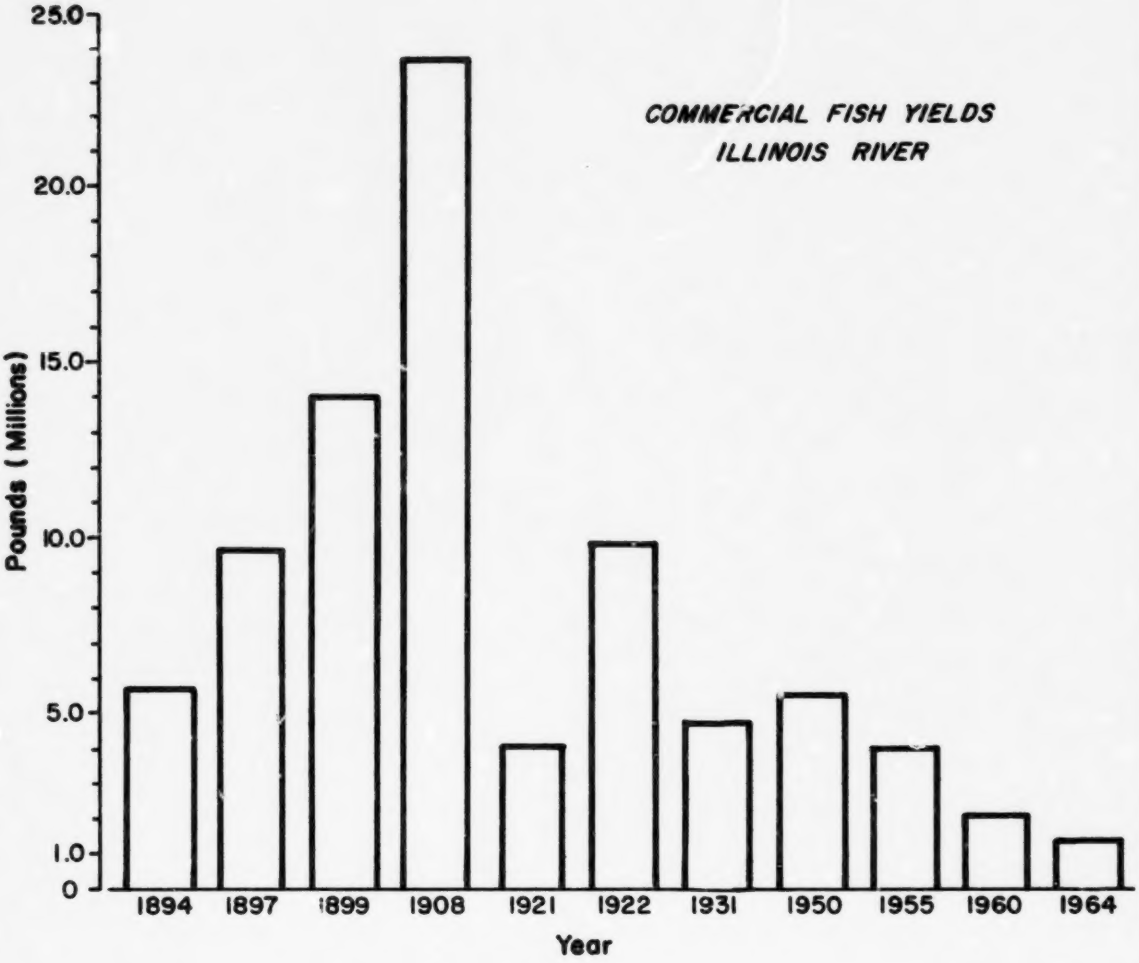

Area covered by Exhibit I
WISCONSIN
ILLINOIS
MILWAUKEE
LAKE MICHIGAN
GALESBURG
Spoon River
Ill. Miss Canal
LASALLE-PERU
OTTAWA
Fox River
AURORA
Ill. Mich Canal
MORRIS
Des Plaines
S. & S. Ship Canal
Chicago R.
River
CHICAGO
JOLIET
Calumet Sag Channel
Kankakee
KANKAKEE
River
MICHIGAN CITY
MICHIGAN
INDIANA
ILLINOIS
INDIANA
Mississippi River
Illinois River
BEARDSTOWN
HAVANA
PEORIA
PEKIN
Mackinaw River
Sangamon River
SPRINGFIELD
GRAFTON
Missouri River
ST. LOUIS
E. ST. LOUIS
40
41
42
43
39
90
89
88
87
86
91
Scale 0 5 10 20 30 40 Miles

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

ILLINOIS RIVER WATERWAY

HARZA ENGINEERING COMPANY DWG. NO. 387P-101
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT- G-3



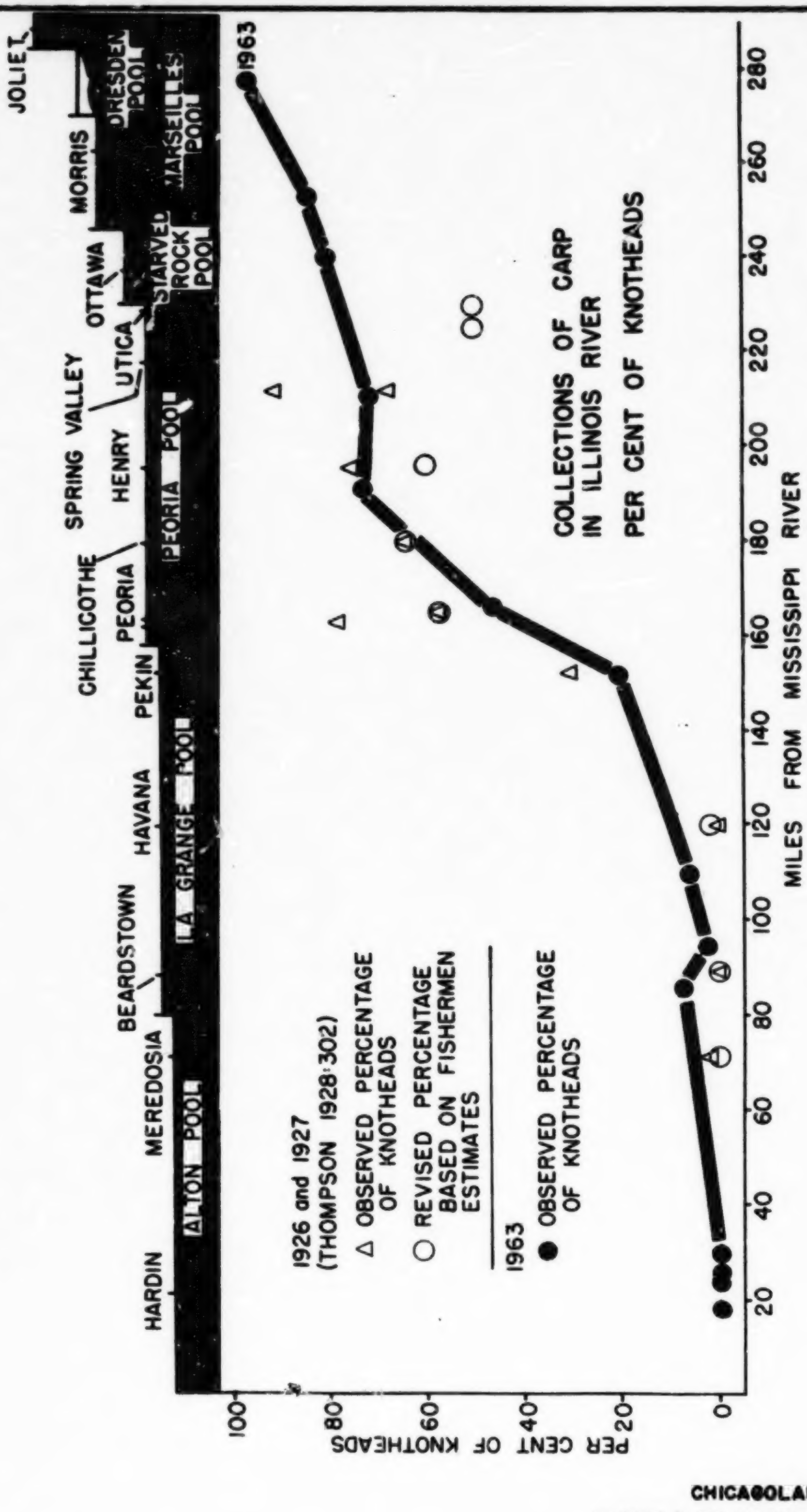
NOTE:
From Illinois Natural History Survey,
Starrett, et al, 1966

CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

COMMERCIAL FISH YIELDS

HARZA ENGINEERING COMPANY DWG. NO. 387P-102
BAUER ENGINEERING, INC. PROJECT 6524

EXHIBIT G-4



CHICAGOLAND AREA
FLOOD & POLLUTION CONTROL STUDIES

COLLECTIONS OF DEFORMED FISH

HARZA ENGINEERING COMPANY DWG. NO. 387P-103
BAUER ENGINEERING, INC. PROJECT 6524